

STEEL- REINFORCED CONCRETE STRUCTURES



Assessment and Repair
of Corrosion

Mohamed A. El-Reedy



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*This book is dedicated to my
mother's spirit, my father, my
wife, and my children.*

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Preface

Deterioration of reinforced concrete structures has been one of the major challenges facing civil engineers over the past 20 years. Concrete structures were famous for being maintenance free. Over time, however, it was found that nothing is free and that it was necessary to increase spending if one did not know how to deal with deterioration of concrete structures. The major factor causing concrete deterioration is corrosion of steel reinforcements, so it is very important to understand this issue thoroughly.

This book aims to be a handbook for reinforced concrete structure maintenance to be used as a guide for junior and senior engineers' work in design, construction, repair, and maintenance. Moreover, the book has been written so as to be easy for civil engineers to understand—without any complicated chemical reactions—and to help researchers with its very strong theoretical background.

The book consists of three main approaches. In the first, chapters 1–3 describe the corrosion phenomena of steel in concrete, the effect of concrete properties on the corrosion, and the precautions taken in construction to control corrosion. The second approach concerns the evaluation of concrete structures and different methods to protect steel reinforcement from corrosion. The last two chapters describe the most traditional and advanced repair techniques. Moreover, chapter 9 focuses on an advanced maintenance plan philosophy as risk-based maintenance for reinforced concrete structures.

This book provides a practical guide to the art of concrete structure repair and maintenance. It includes case histories from all over the world to assist the reader in appreciating the widespread applications and ranges of advanced repair techniques.

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The Author

Mohamed A. El-Reedy's background is in structural engineering. His main area of research is the reliability of concrete and steel structures. He has provided consulting to different oil and gas industries in Egypt and to international companies such as the International Egyptian Oil Company (IEOC) and British Petroleum (BP). Moreover, he provides different concrete and steel structure design packages for residential buildings, warehouses, and telecommunication towers. He has participated in Liquefied Natural Gas (LNG) and Natural Gas Liquid (NGL) projects with international engineering firms. Currently, Dr. El-Reedy is responsible for reliability, inspection, and maintenance strategy for onshore concrete structures and offshore steel structure platforms. He has performed these tasks for hundreds of structures in the Gulf of Suez in the Red Sea.

Dr. El-Reedy has consulted with and trained executives at many organizations, including the Arabian American Oil Company, BP, Apachi, Abu Dhabi Marine Operating Company (ADMA), the Abu Dhabi National Oil Company, King Saudi's Interior Ministry, Qatar Telecom, the Egyptian General Petroleum Corporation, Saudi Arabia Basic Industries Corporation (SAPIC), the Kuwait Petroleum Corporation, and Qatar Petrochemical Company (QAPCO). He has taught technical courses about repair and maintenance for reinforced concrete structures and steel structures worldwide, especially in the Middle East.

Dr. El-Reedy has written numerous publications and presented many papers at local and international conferences sponsored by the American Society of Mechanical Engineers, the American Concrete Institute, the American Society for Testing and Materials, and the American Petroleum Institute. He has published many research papers in international technical journals and has authored four books about total quality management, quality control and quality assurance, economic management for engineering projects, and repair and protection of reinforced concrete structures. He received his bachelor's degree from Cairo University in 1990, his master's degree in 1995, and his Ph.D. from Cairo University in 2000.

1 Introduction

The civilization of any country is measured by the advanced techniques used in constructing its buildings. Concrete is the main element used in construction materials and its development in any country means that there have been developments in that country's engineering thinking. Ancient Egypt used concrete in its buildings and temples; Egyptians used crushed stone as an aggregate and clay as an adhesive. The Greeks used concrete in their buildings and called it *Santorin Tofa* (El-Arian and Atta 1974); the Romans used a material called *pozzolana* that was like concrete. After that, the formula for concrete was lost for many centuries. In the eighteenth century, a number of famous scientists worked to create a formula for concrete:

- John Smeaton used concrete to construct the Eddystone Lighthouse.
- Joseph Parker conducted research on stone and its uses in concrete.
- Edgar researched using cement made from limestone and clay.
- Louis Vicat also tried to develop cement from limestone and clay.
- Joseph Aspdin finally finished his research by developing portland cement.

At the end of the nineteenth century and the beginning of the twentieth, the architectural shapes of buildings underwent major changes as architectural engineers and builders changed their points of view. Previously, they had used the same ideas found in the European Renaissance, using columns and arches to move toward the functional use of buildings. For functional use and to achieve the architectural dream with an economic approach, concrete was the best solution.

Nowadays, reinforced concrete is considered the most famous and important material in the construction industry. It is used for buildings and for different types of civil engineering projects, such as tunnels, bridges, airports, and drainage and hydraulic projects. Research is focused on increasing the performance of concrete to match these varieties of applications.

Reinforced concrete is considered inexpensive compared to other building materials; thus, it has been used for high-rise building for megaprojects and also for smaller projects such as one-story buildings; all these projects are handled by contractors and engineers with different capabilities. Therefore, sometimes the reinforced concrete does not perform correctly according to specifications; in some cases, there is a difference between the quality required by the standard or code and that found on the site. These situations are usually due to a lack of worker, foreman, or engineer competence. Other construction errors are due to a lack of quality-control procedures, materials received at the site that are incompatible with the standard, improper selection of materials, or a concrete mix that does not fit the surrounding environmental conditions.

The main challenge that faces the engineer working in the concrete industry today is the corrosion of steel reinforcement bars. When concrete was first used, concrete structures were traditionally treated as structures without maintenance

needs during their lifetime. For many years it has been obvious that this viewpoint is false, although concrete is indeed a durable material. Our dreams have broken on the rock of truth as we have found that a number of concrete structures have undergone major deterioration because of different factors and that 80% of the deterioration of reinforced concrete structures is due to corrosion of the steel reinforced bars. Based on this fact, much research and many development techniques have focused on providing materials and new methodology to protect steel reinforcement bars from corrosion.

Because severe climatic conditions comprise the major factor causing corrosion in reinforcement steel bars, all specifications and codes provide some cautions for concrete mix design, material selection, concrete cover thickness, and others in order to maintain concrete durability during its lifetime and to prevent corrosion on the steel bars based on different environmental conditions. Corrosion of steel is the enemy of any country's investment in real estate; some structures are destroyed completely due to corrosion and depleted steel bars have a direct impact on the safety of a structure.

Billions of dollars have been spent on the corrosion problem worldwide. For example, the United States has spent \$150 billion to repair bridge decks. In North Africa, near the coast, and in the Middle East in the Arabian Gulf area, some buildings have been destroyed completely due to deterioration of the structures as a result of corrosion in the steel reinforcement bars.

It is important to know the causes of corrosion, the concrete parameters, and the environmental conditions surrounding the structure that cause the start of corrosion and affect the corrosion rate. For instance, it is acceptable in European and Middle Eastern countries to use seawater in the concrete mix. Also, in 1960, calcium chloride was used as an additive to assist in concrete-setting acceleration, and seawater was used in concrete mix until 1970. After that, much damage to the concrete structure was found over a 20-year period—especially in Middle Eastern countries where, because potable water is very expensive, they use seawater in most of the concrete mixes for different structures. This period had a very bad impact on concrete structures from an economic point of view because the construction investment was destroyed. From these facts, the risk of corrosion and its impact on the national income of any country become clear; therefore, it is important to understand corrosion characteristics and causes of corrosion, which are discussed, respectively, in chapters 2 and 3.

Usually, corrosion of steel bars is indicated by a brown spot on the concrete surface; in some cases, cracks parallel to the steel bars will be seen and, in the worst case, the steel bars can be seen directly because the concrete cover has failed. When corrosion occurs, its major impact on structure safety will be the danger of the concrete cover falling, as is the case when the structure loses its strength due to reduction on the concrete cross-section dimension and also reduction on the cross-section of the steel bars due to corrosion. The spalling of concrete cover happens as the steel bars with corrosion increase their volume, with the effect of putting high stress on the concrete cover; this causes cracking and then the cover falls.

Usually, an engineer is requested to assess an existing building, so the theoretical and practical method of performing structure assessment must be known to

structural engineers. This will be discussed in chapter 4. All the costs for repair and reconstruction have a higher impact on a country's economy in general because the money paid to repair and reconstruct could go to other investments.

Chapter 5 discusses different codes and standards that deal with corrosion and provides precautions for design and construction to reduce the risk of corrosion on a structure. Corrosion control is illustrated in chapter 6. Chapter 7 provides all the steps of repair based on standards, all the traditional and advanced techniques to repair, and ways to choose the best method of repair. There are many research and development techniques to protect steel reinforcement bars; all the practical methods available on the market will be illustrated in chapter 8.

During the last 20 years, some researchers have studied the reliability of reinforced concrete structures and have provided different techniques, such as qualitative and quantitative risk assessments. In addition, the maintenance strategy and plan have been reviewed in different studies, based on risk-based maintenance. The target is to reduce maintenance cost as much as possible by using optimization methodology that allows lower maintenance costs, as well as maintaining the structure within a reliability limit that can be functionally used with user satisfaction. All the methods that will assist in implementing the maintenance plan from an economic point of view, as well as the economic method used to choose among steel protection alternatives, will be discussed in the final chapter.

REFERENCE

El-Arian, A. A., and A. M. Atta. 1974. Concrete technology. In *World Book Encyclopedia*.

2 Corrosion of Steel in Concrete

2.1 INTRODUCTION

In the past, most of the design studies in the literature and research in reinforced concrete assumed that the durability of reinforced concrete structures could be taken for granted. However, many reinforced concrete structures are exposed during their lifetimes to environmental stress (e.g., corrosion and expansive aggregate reactions), which attacks the concrete or steel reinforcement (Cady and Weyers 1984; Kilaeski 1980; Mori and Ellingwood 1994; Takewaka and Matsumoto 1988; Thoft-Christensen 1995). Corrosion of reinforced steel bars is considered to have been the most famous problem facing structural engineers in the last decade. In order to avoid corrosion of the steel bars in a concrete structure, we need to understand the corrosion phenomenon so that we can provide different engineering solutions to this problem.

Because corrosion of steel bars in reinforced concrete structures results in great expense, it is mainly considered an economic rather than an engineering problem. In the United States, the cost of repair due to corrosion of steel bars in buildings and bridges is around \$150 million per year. During cold weather when ice accumulates on bridges in the United States, Canada, and Europe, these countries usually use salt to melt the ice. However, as we will discuss, salt contains sodium chloride, which is a main source of steel corrosion in these countries, so the cost of maintaining and repairing bridge decks is very high. A transportation research center report in 1991 indicated that the cost of bridge deck repair was from \$50–200 million per year; for some parts of bridges, repair costs were around \$100 million per year. Repairs in multifloor parking garages cost \$50–150 million per year. In England and Wales, bridge repair due to corrosion cost £616.5 million in 1989. Considering that these two countries have only 10% of the United Kingdom's bridges, imagine the impact of corrosion on countries' economies!

On the other hand, in the Middle East, especially in the gulf area, countries that are the main producers of oil and gas worldwide have very strong economies. The result is that it is possible to make huge investments in construction projects, which are the main parts of any developing country's growth. The bad news is that all these countries have severe environmental conditions due to high temperatures, which can reach 55–60°C. In addition, because most of these countries are located near the Arabian Gulf, they have a high relative humidity and the ground water has a high percentage of salt.

All these factors increase the probability of corrosion of steel bars. From the quality point of view, a higher temperature prevents a very good curing process. Thus, all these factors accelerate the deterioration of concrete structures in these

countries and cause their buildings to have short lifetimes—especially those constructed in the period between 1970 and 1980 (before the practice of using additives in concrete). Therefore, in the Middle East, the amount of money lost due to corrosion of steel bars is very high; this is considered a challenge to engineers to use new methods and techniques to protect these bars.

The corrosion process occurs slowly and propagates with time, so the deterioration rate varies. Corrosion on steel bars affects a structure's safety, which depends on the surrounding environmental conditions that mainly affect the corrosion rate, the location of the member in the building, and the type of the member.

When will inspection and repair be performed? This is a big question that needs an accurate answer because reducing the time between inspections will cost, but will maintain the structure safety. On the other hand, increasing the time between inspections can decrease cost, but will affect structure safety. Many accidents have happened due to this policy; for example, a large part of a bridge in New York fell, causing a motorcycle driver to lose control of his vehicle and die. It is obvious that carelessness with regard to the corrosion process may result in death.

Figure 2.1 shows a clear example of the effect of corrosion on concrete structure deterioration. It shows cracks and the concrete cover and parts of the plaster are falling. This reinforced concrete beam is in a villa near the coast of the Mediterranean Sea.

In 1960, in Europe and the Middle East, it was acceptable to use seawater in the concrete mixture, as well as to use calcium chloride additives as a concrete setting accelerator; these practices continued until 1970. In the Middle East, pure water is very expensive, so they have used seawater extensively; after 20 years, they have found severe deterioration on reinforced concrete structures, causing many problems from an economic point of view (i.e., construction investments).

From the preceding discussion, it is obvious that the effect of corrosion not only creates a serious problem for structural engineers but also affects countries' national incomes. Therefore, engineers must have an in-depth understanding of the nature of the corrosion process and the reasons for its occurrence. Understanding the problem is the first step in solving the problem.



FIGURE 2.1 Deterioration in a reinforced concrete beam.

In this chapter, the main principle of corrosion will be discussed—especially corrosion of the steel reinforcements in concrete. How and why corrosion occurs will be discussed. There are many ways to illustrate the corrosion process in concrete; we will illustrate it by the first principle, which will match with engineering studies; we will try to stay away from chemical equations. Understanding the corrosion process and how it occurs will be the first step to knowing how to protect steel bars from corrosion and to differentiate among the different protection methods that will be illustrated later.

If we put a nail in dry air or immerse it in water, it will not corrode. However, if it is immersed in water and then put in the air, it will corrode. This happens for any type of normal or high-strength steel. In the case of steel embedded in concrete, the concrete is a porous material containing water in the void due to the process of curing or because of rainy weather or any weather with high relative humidity. Thus, the concrete will contain humidity, which is a common cause of corrosion. The good news is that it is not necessary to corrode the steel bars embedded in concrete because concrete is alkaline in its nature and alkalinity is opposite to acidity. As a result of its alkalinity, concrete can protect the steel bar from corrosion. Why is concrete alkaline? Inside the microvoid of the concrete is a high concentration of the oxides calcium, sodium, and magnesium.

These oxides produce hydroxides that have a high alkalinity when water is added (pH 12–13). pH is the measure of acidity and alkalinity and is based on the percentage of hydrogen ion concentration with respect to hydroxide ions; the maximum acidity concentration occurs when pH = 1 and high alkaline concentration is present at pH = 14. This alkalinity will produce a passive layer on the steel reinforcement surface; consisting of oxides and hydroxides for iron and part of cement, this layer is dense and prevents the occurrence of corrosion. This layer will not remain for long. Two factors always have an effect in breaking it: the carbonation process and the permeability of chlorides to the steel reinforcement. This will be illustrated in detail in chapter 3; here, the general corrosion process will be illustrated without considering any external factors' effects on the start of the corrosion process.

2.2 THE CORROSION PROCESS

After the passive layer is broken down, rust will appear instantly on the steel bar's surface. The chemical reactions are the same in cases of carbonation or of chloride attack. When the corrosion of the reinforced steel bars in concrete occurs, they melt in the void that contains water. The electrons will accumulate according to the following equation, which presents the *anodic reaction*:

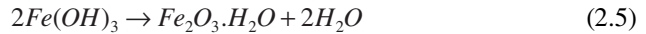
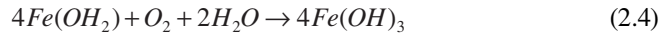


If the electron will be accumulated on the other part of the steel reinforcement but cannot accumulate with huge numbers in the same location, there is another reaction that uses the number of the electrodes with oxygen and water—the *cathodic reaction*. Its equation is:



From this equation it is found that the presence of OH^- occurs due to cathodic reaction. The hydroxide ions increase the alkalinity and reduce the effect of carbonates or chlorides slightly. From this equation, it is important to know that water and oxygen are the main reason for the corrosion process.

As shown in the preceding equations and figure 2.2, the anodic and cathodic reactions are the first step in the process of producing corrosion as the hydroxide ions (OH^-) will react with ferrous iron (Fe^{2+}) as a result of chemical equation (2.1). This reaction will produce ferrous hydroxide, which will react with oxygen and water again and produce ferric hydroxide. This chemical reaction is shown graphically in figure 2.2.



The preceding chemical reactions show the transformation of steel from ferrous hydroxides ($Fe(OH)_2$), which will react with oxygen and water to produce ferric hydroxides ($Fe(OH)_3$), and the last component, which is the hydrate ferric oxide (rust); its chemical term is $Fe_2O_3 \cdot H_2O$.

Ferric hydroxide has more effect on concrete deterioration and spalling of concrete cover as its volume will increase the volume of the original steel bars by about two times or more. When iron goes to hydrated ferric oxides in the presence of water, its volume will increase more to reach about 10 times its original volume and will become soft. In this stage, cracks on concrete start until the concrete cover falls; rust, with its brown color, can clearly be seen on the steel bar.

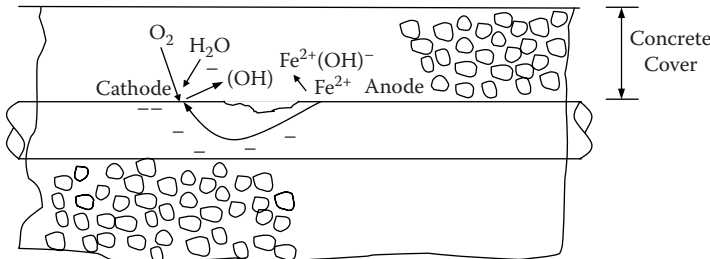


FIGURE 2.2 Corrosion process on a steel reinforcement surface.

2.3 BLACK CORROSION

This type of corrosion occurs when there is a large distance between anode and cathode locations and also if oxygen is not available. This usually occurs in cases of buildings immersed in water or when a protective layer prevents presence of oxygen. It is called black corrosion because, when it cracks, the bars will have a black or green color. This type of corrosion is critical and causes a problem because it does not provide any warning of cracks or falling concrete cover when corrosion occurs. This case occurs usually for offshore structures with parts submerged in water or for a concrete structure partially covered by a waterproof membrane or coating.

2.4 PIT FORMATION

Corrosion in steel bars starts by forming a small pit. After that, the number of pits will increase with time and then the combination of these pits causes a uniform corrosion on the surface of the steel bars. This is obvious in the case of a steel reinforcement exposed to carbonation or chloride effects, as shown in figure 2.3. The pit formation is shown in figure 2.4.

Many chemical reactions describe the formation of pits and, in some cases, these equations are complicated. But the general principle of pit corrosion is very simple, especially in cases of chloride attacks. In some locations on the steel reinforcement, voids for the cement mortar are present around the steel reinforcement or sulfide is present inside steel bars so that the passive layer is more vulnerable to chloride attack; an electrochemical potential difference attracts chloride ions. Corrosion is initiated and acids are formed as hydrogen from the supplied MnS inclusion in steel and HCL from the chloride ions, if they are present. The following chemical reactions are simple representations of this process:

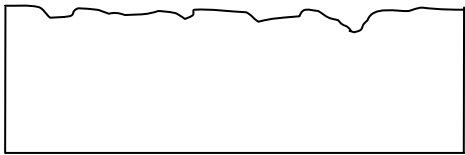
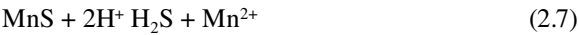


FIGURE 2.3 Uniform corrosion.



FIGURE 2.4 Pitting corrosion.

Rust may form over the pit, concentrating the acid (H^+) and excluding oxygen so that the iron stays in solution, preventing the formation of a protective oxide layer and accelerating corrosion. We will return to the subject of pitting corrosion later. It is related to the problems of coated reinforcement and to the black rust phenomenon that will be discussed.

This corrosion state is characterized by galvanic action between a relatively large area of passive steel acting as cathode and a small anodic pit where the local environment inside the pits has a high chloride concentration and decreased pH value. For pitting to be sustained, it is necessary that a reasonable amount of oxygen should be available to cause polarization of the anode. The average corrosion potential of steel reinforcement has a pitting that is likely to vary between that of the passive state and that of the anodic pitting areas—typically in the range of -200 to -500 mV.

2.5 BACTERIAL CORROSION

Bacteria are another cause of corrosion. Because bacteria exist in soil, the foundation is considered the main element exposed to this type of corrosion. These bacteria will convert sulfur and sulfides to sulfuric acid. The acid will attack the steel and then cause initiation of the corrosion process. Other bacteria that attack the sulfide exist in the steel reinforcement FeS due to reactions. This type of corrosion is often associated with a smell of hydrogen sulfide (rotten eggs) and smooth pitting with a black corrosion product when steel bars are exposed to soil saturated with water.

From the preceding discussion, it can be seen that any cracks or honeycombs in reinforced concrete foundations must be repaired by qualified materials before refilling with sand and compacting. As we will discuss in the following chapters, the repair of foundations is very complicated and very expensive. Therefore, avoiding these defects in construction will be very important to maintaining a structure along its lifetime.

2.6 CORROSION RATE

The corrosion process and its shapes have already been described. In this section the corrosion rate of the steel reinforcement will be discussed. The corrosion rate is considered the most important factor in the corrosion process from a structural-safety perspective and in the preparation of the maintenance program for the structure. This factor is considered an economic factor of structural life. When the corrosion rate is very high, the probability of structure failure will increase rapidly and structural safety will be reduced rapidly. The corrosion rate depends on different factors, so if we can control these factors, the corrosion rate will be slow. The corrosion will occur, but it will not cause a serious problem to the structure if the rate is low.

The main factor that affects the corrosion rate is the presence of oxygen, especially in the cathodic zone shown in the previous chemical reactions and in figure 2.5. In a case of no oxygen, the corrosion rate will be slow and different methods are used to prevent the propagation of the oxygen inside the concrete—for example, to take care of the concrete compaction in order to obtain a dense concrete cover so that propagation of the oxygen will be very slow or preventive. Practically speaking,

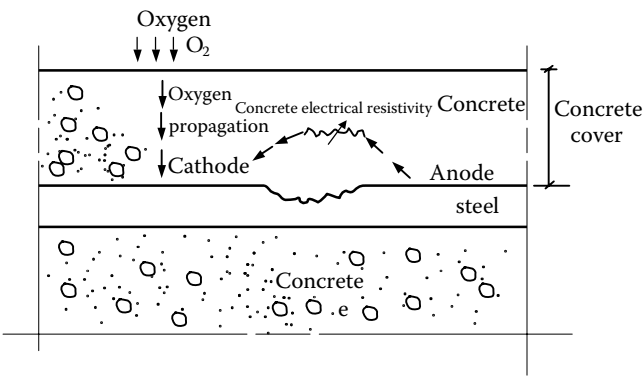


FIGURE 2.5 Factors affecting corrosion rate.

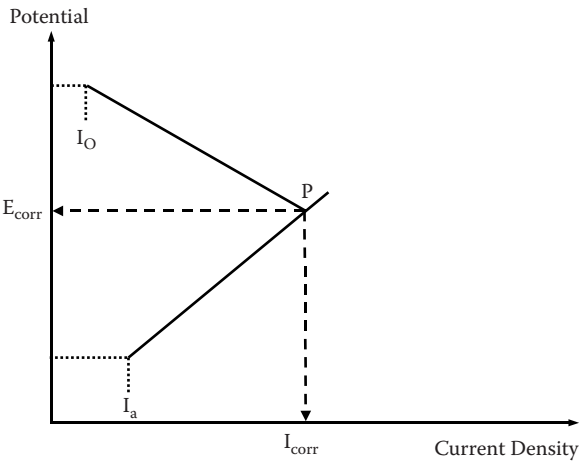


FIGURE 2.6 Evans diagram for a simple corrosion couple.

this is an ideal case that cannot happen but that we try to reach. When we prevent the propagation of oxygen inside the concrete, the oxygen at the steel bars will be less. The difference in volts between anodic and cathodic zones will also be less—an effect on corrosion called “polarization.” The Evans diagram in figure 2.6 shows the polarization curves for separate anodic and cathodic reactions intersecting at point (P), where the mean anodic and cathodic current densities are equal and represent the corrosion rate in terms of a mean corrosion current density, I_{corr} . The electrode potential of the couple at this point is termed the corrosion potential, E_{corr} .

The second important factor affecting the corrosion rate is the moving of the ions inside the concrete voids around the steel reinforcement. If the speed of moving ions is very slight or prevented, the corrosion rate will be very slow also or, in the ideal case, prevented. This case may happen when the concrete around the steel bars has a high resistance to electrical conductivity between anode and cathode.

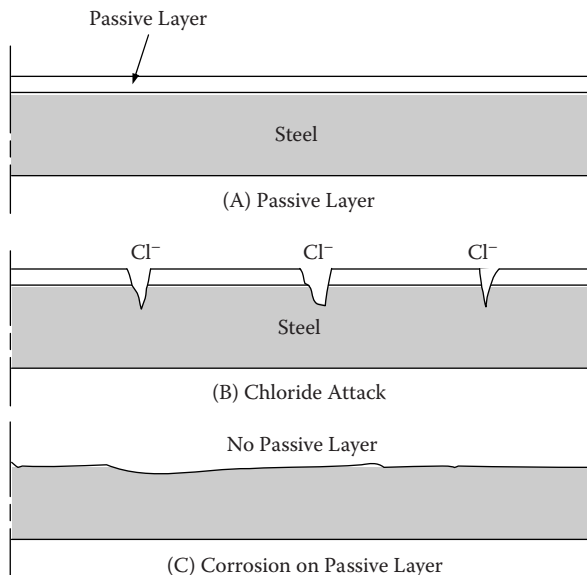


FIGURE 2.7 Effect of chlorides on demolishing the passive layer.

The measurement of electrical resistivity to the concrete surrounding the steel reinforcement can give us an assumption of the corrosion rate and the chemical reaction rate. There are many ways to measure corrosion rate by measuring the electrical resistivity, as will be described in detail in chapter 4.

Another factor can control the corrosion rate because there are different types of metal. One of them is steel reacting with oxygen and forming a layer with $0.01\ \mu\text{m}$ from oxides from the same metal at the surface, if this layer is stable, and protecting it from breaking in any circumstances. The metal is considered protected when it is inside the liquid solution and reduces the melting of ions in the liquid solution surrounding the steel bars so that the corrosion rate will be very slow if this passive layer exists. The corrosion rate will be neglected and considered zero (shown in fig. 2.5).

In this case, this layer will be the passive protection layer to the steel reinforcement. The protected passive layer is first responsible for protection from corrosion, and this is obvious in the case of stainless steel. The steel in this case consists of some chromium and nickel and other metals that improve the stability of the protected passive layer. The passive protection layer will be stable if the aqueous solution contains a high quantity of hydroxide ions (OH^-).

This layer is affected by chloride ions (Cl^-) or carbonates reducing the hydroxide in the water solution in the concrete void that will assist in formation of voids and pitting in steel, as shown in figure 2.7. After extensive pitting takes place, they will combine to destroy the passive layer and start the rusting process. In general, the state of corrosion of steel in concrete may be expected to change as a function of time. In attempts to model this time-dependent corrosion behavior, it is convenient to distinguish and understand clearly the following stages:

- During the initiation period, from day 1 of construction until the steel bars have remained passive in the protected layer within the concrete, environmental changes are taking place that may ultimately terminate passivity.
- The corrosion period begins at the moment of depassivation and involves the propagation of corrosion at a significant rate until the third step is reached.
- The final stage is reached when the structure is no longer considered acceptable on grounds of structural integrity, serviceability, or appearance.

2.7 SUMMARY

This chapter has illustrated the corrosion process, reasons for its occurrence, accompanying chemical reactions, electron movement, and the formation of oxides. It is important to understand the shape of corrosion and its type as pitting formation, black corrosion, and bacterially induced corrosion. Understanding anode and electron formation is important in order to know how to protect steel bars.

Other chapters will discuss how corrosion can happen and the factors that affect it. Understanding corrosion is a very important step to obtaining engineering solutions to avoid it, as well as obtaining the best methods for protection and repair of reinforced concrete structures exposed to corrosion. You must know your enemy before you can fight him.

It is important to mention that the quality of concrete and the thickness of the concrete cover to the steel reinforcement affect the stability of the passive layer. They influence the ability of the system to exclude aggressive substances, which tend to alter pore-water composition in ways that endanger the passivity of embedded steel and thus induce significant corrosion.

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3 Causes of Corrosion and Concrete Deterioration

3.1 INTRODUCTION

The process of steel corrosion has been discussed generally in the previous chapter, as well as how it occurs and key concepts applicable to it. This chapter will discuss different kinds of mechanical corrosion of steel in concrete structures. There are two main reasons for corrosion of steel in concrete: chloride attack and carbon dioxide penetration, which is called the carbonation process. These two processes cause corrosion of steel, but they do not influence the corrosion of steel in concrete directly. Rather, there are other reasons, such as the presence of certain chemicals inside the concrete and voids that affect the steel. Moreover, some acids, such as sulfate, will attack the concrete and cause concrete deterioration and corrosion of steel and then break the concrete alkalinity around the steel bars.

When acids attack the concrete, this causes problems such as the impact on steel reinforcement. The main factors that cause the corrosion in reinforced concrete structures exposed to carbonation or chlorides will be discussed here, as well as the basics of corrosion and how they apply to steel in concrete, corrosion rate, and corrosion effects on spalling of concrete.

Concrete is alkaline, which is the opposite of acidic. Metals corrode in acids, whereas they are often protected from corrosion by alkalis. Concrete is alkaline because it contains microscopic pores with high concentrations of soluble calcium, sodium, and potassium oxides. These oxides form hydroxides derived from the reactions between mixing water and portland cement particles, which are very alkaline. A measure of acidity and alkalinity, pH, is based on the fact that the concentration of hydrogen ions (acidity) times hydroxyl ions (alkalinity) is 10^{-14} mol/L in aqueous solution. A strong acid has pH = 1 (or less), a strong alkali has pH = 14 (or more), and a neutral solution has pH = 7. Concrete has a pH of 12–13; steel starts to rust at pH 8–9.

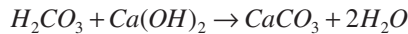
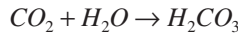
Concrete creates a very alkaline condition within pores of the hardened cement mix that surrounds aggregate particles and the reinforcement. The alkaline condition leads to a “passive” layer on the steel surface. A passive layer is a dense, impenetrable film that, if fully established and maintained, prevents further corrosion of the steel. The layer formed on steel in concrete is probably part metal oxide/hydroxide and part minerals from the cement. A true passive layer is a very dense, thin layer of oxide that leads to a very slow rate of oxidation (corrosion). Once the passive layer breaks down, areas of corrosion will start appearing on the steel surface. The chemical reactions are the same whether corrosion occurs by chloride attack or carbonation.

The main causes of corrosion of steel in concrete are chloride attack and carbonation transformation. These two mechanisms are unusual in that they do not attack

the integrity of the concrete. Instead, aggressive chemical species pass through the pores in the concrete and attack the steel. This is unlike normal deterioration processes due to chemical attack on concrete. Other acids and aggressive ions such as sulfate destroy the integrity of the concrete before the steel is affected. Most forms of chemical attack are therefore concrete problems before they are corrosion problems. Carbon dioxide and chloride ions are very unusual in penetrating the concrete without significantly damaging it. Accounts of acid rain causing corrosion of steel embedded in concrete are unsubstantiated. Only carbon dioxide and the chloride ion have been shown to attack the steel and not the concrete.

3.2 CARBONATION

Carbonation is the result of the chemical reaction between carbon dioxide gases in the atmosphere and the alkaline hydroxides in the concrete. Like many other gases, carbon dioxide dissolves in water to form an acid. Unlike most other acids, the carbonic acid does not attack the cement paste, but rather neutralizes the alkalis in the pore water, mainly forming calcium carbonate:



Calcium hydroxyl exists in the concrete and increases the concrete alkalinity that maintains a pH level of 12–13; after carbonates attack inside the concrete and spread, it will form calcium carbonate. As seen in the previous equation, from this the value of pH will be reduced to the level that causes the corrosion to the steel reinforcement, as shown in figure 3.1.

The carbonation process occurs quickly when the concrete cover is not very thick. It may also occur when the concrete cover over the steel bars is thick because the carbonation transformation will happen as a result of the existence of pore voids open in the concrete that assist the quick propagation of CO_2 inside the concrete. The carbonation process can occur when the alkalinity in voids is relatively small. This happens when the cement content is small and water-to-cement (w/c) ratio is high and also due to a bad curing process during construction.

Carbonation moves inside concrete according to the diffusion theory: The diffusion rate is in inverse proportion to the distance between the steel bars and the concrete surface, which is the concrete cover thickness:

$$\frac{dx}{dt} = \frac{D_o}{x} \quad (3.1)$$

where

x = the distance from the concrete surface

t = the time

D_o = the diffusion rate, which depends on the quality of the concrete

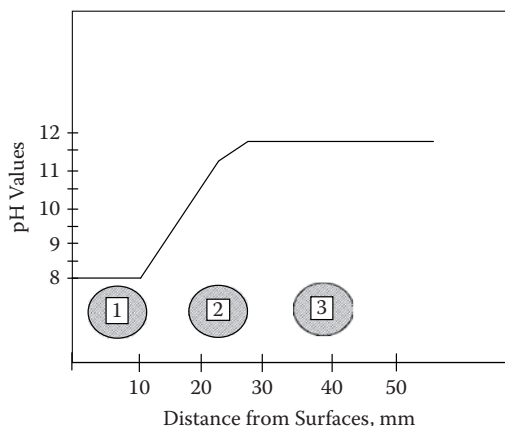


FIGURE 3.1 Relation between carbonation depth and level of pH values.

When the concrete is affected by carbonation, the value of alkalinity will drop from pH 11–13 to pH < 8, as shown in figure 3.1. At this level of alkalinity, the passive layer protection cannot protect steel, so the corrosion process will start.

Many factors affect the ability of concrete to resist the spread of carbonation in the concrete. Note that the rate of carbonation depends on the thickness of the concrete cover and also on its quality in terms of the mixing ratios that achieve the highest quality. This is necessary to resist the spread of carbonation inside concrete.

When the concrete is of high quality and the process of compaction as well as the curing process has been well done, it would be a difficult task to let CO_2 spread inside the concrete. As we mentioned, if the compaction is good enough to reduce the voids in concrete, it will enhance the protection of the concrete from spread of carbon dioxide; this can be enhanced by using new materials such as silica fume, which has a very fine particle size of around 0.1–0.2 μm that will reduce the voids in concrete. Silica fume with plasticizer is used now to produce high-strength concrete that can provide concrete compressive strength with around 120 N/mm^2 and more. Moreover, in mid-1980s the Japanese produced concrete without compaction with a high percentage of silica fume and special concrete mix.

Different specifications and codes have identified an appropriate thickness for the concrete cover, depending on the nature of the environmental conditions surrounding the structure. As well, they have identified the w/c ratio in concrete mix, reasonable cement content, the suitable compaction method, and the curing processing time required to avoid a process of carbonation transformation to the concrete. Chapter 4 will explain the various codes in detail.

The carbonation process is the main reason for corrosion of reinforcing bars in old structures, those that have been poorly constructed, or structures containing a small proportion of cement content in the concrete mix. The carbonation process rarely occurs in modern concrete bridges and new buildings, where the proportion of water to cement is low, cement content is sufficient, and the compaction and the concrete curing are very good. Also, in these structures the concrete cover will be

sufficient to prevent the spread of carbonation to the steel reinforcement throughout the structure’s lifetime.

In structures that are exposed to seawater or dissolved salt, chlorides in concrete often penetrate until they reach the steel reinforcement bars; they cause a steel corrosion that is faster than the impact of the process of carbonation. The cycle of dry and wet on the surface of concrete helps speed the process of carbonation; entry of carbon dioxide gas is permitted in the dry cycle and dissolved in the wet cycle. This is considered a problem in some coastal cities exposed to cycles of wet and dry, such as Hong Kong.

If, upon investigation, it is found that corrosion has occurred as a result of inadequate concrete cover thickness, then the concrete around the steel reinforcement is carbonated and causing the corrosion. When the concrete cover is not very thick, the process of carbonation transformation will speed up. This situation may lead to the process of corrosion beginning 5 years from the year of construction. When the concrete is of good quality, the carbonation will find it difficult to spread and the inadequate thickness of the concrete cover will not have a huge effect.

Carbonation can be observed and measured easily. The pH indicator is the proportion of phenolphthalein solution in water and alcohol, which will be sprayed or painted on the concrete surface that is expected to be affected by carbonation; phenolphthalein will change its color by change of pH. Phenolphthalein is colorless when pH is low (the carbonation). It changes to the color pink when the pH increases (concrete without carbonation). This can be measured by taking samples of concrete (usually the part of the cover that falls) so that the surface will be ready for the test or to crack part of the concrete when periodic maintenance is performed. The surface to be measured by the amount of pH should be clean and away from dust and other fine materials. From figure 3.1, it can be seen that there is a different pH when carbonation is present in or absent from concrete; this test will be explained in chapter 5.

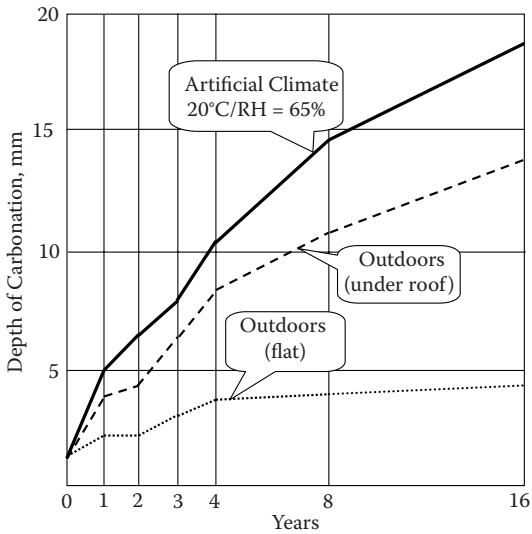


FIGURE 3.2 Effect of climatic conditions on the rate of carbonation.

3.3 SPREAD OF CARBONATION INSIDE CONCRETE

Spread of carbon dioxide inside the concrete and the rate of movement of carbonation almost follow Fick's law in such circumstances that the diffusion rate is in inverse proportion to the distance from the surface, as in the previous equation. Since the carbonation process is considered a change in the characteristics of pore voids in concrete, reduces the base, and reduces the percentage of pH, this equation is approximate. Each of the divisions (one of the characteristics of concrete), the change in the components of concrete, and change of humidity level with depth will deviate from the values of diffusion calculated from the previous law, as in the previous equation.

By performing integration in equation (3.1), we will obtain the second root that assists in defining the carbonation movement. This equation is used with some research to obtain some of the equations that determine the relationship between the rate of carbonation spread and the quality of concrete and environmental factors surrounding the structure. Table 3.1 summarizes some of those equations and explains the transactions included in them.

However, because the carbonation process modifies the concrete pore structure as it proceeds, this is only an approximation. Cracks change in concrete composition and moisture levels with depth will also lead to deviation from the perfect diffusion equation. Integration of equation (3.2) gives a square root law that can be used to estimate the movement of the carbonation front.

Empirically, a number of equations have been used to link carbonation rates, concrete quality, and environmental situations. Table 3.1 summarizes some of those equations and shows the factor that has been included. Generally, there is time dependence. As discussed earlier, the other factors are exposure, w/c ratio, strength, and CaO content (functions of cement type and its alkali content).

For example, consider the basic equation:

$$d = At^n \quad (3.2)$$

where

d = the carbonation depth in millimeters

a = the coefficient

t = the time (years)

n = an exponent, usually equal to 0.5

A number of empirical calculations have been used to derive values of A and n based on such variables as exposure conditions (indoor and outdoor, sheltered or unsheltered), 28-day strength, and w/c ratio as shown in table 3.1.

Schiessl (1988) shows the relation between time and the depth of corrosion at different environmental conditions (see fig. 3.2). There are three curves. The first one is under laboratory conditions with a temperature of 20°C and relative humidity (RH) of about 65%, which gives a higher carbonation depth with time. The second curve puts the concrete outside the laboratory, but under a roof, which provides shade; the

TABLE 3.1
Equations to Calculate Carbonation Depth

Equations

$$d = A(t)^n$$

$$d = ABC t^{0.5}$$

$$d = A(Bw/c - C)t^{0.5}$$

$$d = 0.43(w/c - 0.4)(12(t - 1))^{0.5} + 0.1$$

$$d = 0.53(w/c - 0.3)(12t)^{0.5} + 0.2$$

$$d = (2.6(w/c - 0.3)^2 + 0.16)t^{0.5}$$

$$d = ((w/c - 0.3)^2 + 0.07)t^{0.5}$$

$$d = 10.3e^{-0.123f/28} \text{ at 3 years}$$

$$d = 3.4e^{-0.34f/28} \text{ at 3 years}$$

$$d = 680(f/28 + 25)^{-1.5} - 0.6 \text{ at 2 years}$$

$$d = A + B/f/28^{0.5} + c/(CaO - 46)^{0.5}$$

$$d = (0.508/f/35^{0.5} - 0.047)(365t)^{0.5}$$

$$d = 0.846(10w/c/(10f/7)^{0.5} - 0.193 - 0.076w/c)(12t)^{0.5} - 0.95$$

$$d = A(T - t_i)t^{0.75} C_1/C_2^{0.5}$$

Coefficients

d = carbonation depth

t = time in years

A = diffusion factor

n = exponent (approximately 0.5)

$A = 1.0$ for external exposure

$B = 0.07$ – 1.0 depending on surface finish

$C = R (w/c - 0.25)/(0.3(1.15 + 3w/c))^{0.5}$

For water cement ratio (w/c) ≥ 0.6

$C = 0.37 R (4.6w/c - 1.7)$ for $w/c < 0.6$

R = coefficient of neutralization, a function of mix design and additives

A is a function of curing

B and C are a function of fly ash used

28-day cured

Uncured

Sheltered

Unsheltered

Unsheltered

fX = strength at day X

Sheltered

CaO is alkali content expressed as CaO

t_i = induction time

T = temperature in kelvins

C_1 = CO_2 concentration

C_2 = CO_2 bound by concrete

Source: Parrot (1987).

third curve gives us less carbonation depth with time when concrete is in a flat area outside that is not under shade.

In general, a variable (t) reflects the time in years. This factor is independent and other factors depend on the weather conditions; concrete strength; w/c ratio; the content of the cement used; and the resistance of concrete and calcium oxide content, which depends on the type of cement. For example, consider the equation:

$$d = A\sqrt{t} \quad (3.3)$$

The depth of carbonation is about 16 mm in 16 years in the case of weak concrete and 4 mm in 20 years for good-quality concrete. Therefore, it can be expected that the diffusion rate ranges between 0.25 and 1 mm/year.

Parrott conducted research work for the time of the beginning of steel bar corrosion; the results are made clear in chapter 4. It is well known that, in bridges, the carbonation is almost nonexistent—even after 20 years—because during the construction of bridges more care in compaction, curing, concrete mix, and concrete quality will be taken. However, residential buildings will not reach 60 mm or more.

Engineers can control the carbonation process by taking into account the proper selection of the thickness of the concrete cover and concrete mix by lowering the w/c ratio. Moreover, attention must be paid to quality control of the construction reinforcement concrete structures. The measurement of permeability is important not only to rely on the measurement of concrete strength but also to provide durable concrete structures that can withstand the carbonation process.

3.3.1 PARROTT'S DETERMINATION OF CARBONATION RATES FROM PERMEABILITY

For a new concrete mix or structure, the prediction of carbonation rate is complicated by the lack of data to extrapolate. In a series of papers (Parrott 1994a,b; Parrott and Hong 1991), a methodology was outlined for calculating the carbonation rate from air permeability measurements with a specific apparatus. Parrott (1987) analyzed the literature and suggested that the carbonation depth D at time t is given by:

$$D = aK^{0.4}t^n - C^{0.5} \quad (3.4)$$

where K is air permeability (in units of 10^{-6} m^2) and c is the calcium oxide content in the hydrated cement mix for the concrete cover; $a = 64$. K can be calculated from the value at 60% relative humidity, r , by the equation:

$$K = mK_{60} \quad (3.5)$$

where $m = 1.6 - 0.0011r - 0.0001475r^2$ or $m = 1.0$ if $r < 60$; n is 0.5 for indoor exposure but decreases under wet conditions to

$$n = 0.02536 + 0.01785r - 0.0001623r^2 \quad (3.6)$$

Therefore, increasing the concrete cover depth is required to prevent carbonation from reaching the steel. The concrete cover depth can be calculated based on the measure of air permeability and relative humidity.

3.4 CHLORIDE ATTACK

Chlorides can attack concrete from more than one source. The first source is from inside the concrete during the casting process; the second is to move concrete from outside to inside. When casting takes place, chlorides exist in concrete as a result of the following:

- using seawater in the concrete mix
- using calcium chloride in additives required to accelerate setting time

- aggregate that contains chlorides that can be washed well
- additives that have a higher chloride content than that defined in the specification
- water used in the concrete mix that has a higher number of chloride ions than that allowed in the specifications

Chlorides can propagate inside concrete from the external environment by:

- concrete exposed to seawater spray or continuous exposure to salt water
- using salt to melt ice
- presence of chlorides in chemical substances that attack the concrete structure, such as salt storage

In most cases, the impact of chlorides is from external sources such as seawater spray or use of salt in melting ice. The effect of chlorides on corrosion occurs very quickly in cases of existing chlorides in the water mixing compared to the effect of chlorides from environmental conditions surrounding the building. This often happens in an offshore structure as the concrete mix may contain seawater.

3.5 CHLORIDE MOVEMENT INSIDE CONCRETE

Similarly to carbonation rate movement, chlorides almost match the previous rule of diffusion. But this situation is more complicated. Salt water is absorbed quickly by dry concrete. In some cases, the movement of saline water by rising capillarity is characteristic and also due to some reactions between the concrete and chlorides, which are absorbed inside the concrete pore voids. The fundamental problem corresponding to the chloride breakthrough is the initial concentration of chlorides when spread in the concrete. Note that the previous equation for calculating the spread of carbonation in concrete cannot be used easily in the case of chlorides. The general shape of the changing chloride concentration with depth in the concrete is explained in figure 3.3.

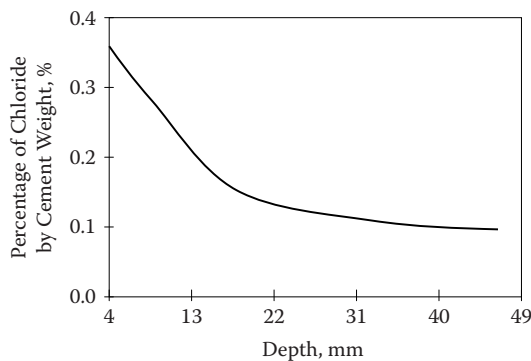


FIGURE 3.3 Chloride concentration at different depths for bridges exposed to the sea.

From this figure we note that the concentration of chlorides is reduced rapidly whenever propagation occurs inside the concrete. It is difficult to identify chloride concentration directly on the surface, where the concentration changes with time from 0–100%, depending on the dry or wet surface, evaporation, and many other factors. Therefore, it is common practice to measure concentration of chlorides about 5 mm from the surface.

From the previous figure, it can be seen that the chloride concentration with depth has a simple parabolic curve and some research is based on this (Nordic Concrete Research Group; Poulsen 1990). It is worth mentioning that the propagation of chlorides inside concrete depends not only on the diffusion factor, but also on other factors influencing the start of the chlorides: the capillarity rise phenomenon and absorption coefficient, which affect the outset at the first 5 mm from the surface.

3.6 CORROSION RATES

The carbonation transformation process, or spread of chlorides inside concrete, is the main reason for breaking the passive protection of steel reinforcement and thus causing the beginning of corrosion. When corrosion starts, the deterioration of steel depends on the rate of corrosion. This, in turn, depends on many factors; in the case of carbonation, the rate of corrosion is greatly influenced by moisture-relative terms. It is much lower during a decrease in the relative humidity inside the concrete voids (less than 75%); the rate of corrosion increases significantly when the relative humidity increases (at 95%), based on Tutti's studies in 1982. The temperature has a big impact on the rate of corrosion. For any sort of corrosion, there is also a reduction (by a factor of approximately 5–10) in corrosion rate with 10°C reductions in temperature.

According to Schiessl (1988), the rate of corrosion in carbonated concrete is a function of RH, wet or dry conditions, and the chloride content. Thus, the decisive parameters controlling corrosion in carbonated concrete will be associated with steady-state RH or wetting/drying cycles and conductivity increases associated with level of chloride in the concrete. However, the corrosion rates found in most research are within 0.015–0.09 mm/year (e.g., research performed in 1992 by El-Abiary et al.).

3.6.1 STATISTICAL ANALYSIS FOR INITIATION OF CORROSION AND CORROSION RATE

The corrosion rate is the main factor driving the maintenance strategy; therefore, to create a risk-based maintenance plan, it is important to know the corrosion rate statistics and also any variables or uncertain factors that affect a concrete structure's reliability. In reliability analysis of concrete structures, many researchers have used different probabilistic models to describe initiation of corrosion and corrosion rate of steel bars in concrete. In 1994, Mori and Ellingwood used the Poisson process with parameters $v(t)$ to describe the initiation of corrosion following carbonation. The mean Poisson ratio is the parameter $v(t)$, expressed as follows:

$$v(w) = \begin{cases} 0 & \text{for } w < t^* \\ v & \text{for } w \geq t^* \end{cases} \quad (3.7)$$

where t^* is a deterministic time, considered to be 10 years, and v is the mean initiation rate of corrosion, which is considered to be equal to 0.2/year.

On the other hand, typical corrosion rates of steel in various environmental situations have been reported in recent years. According to Ting (1989), the average corrosion rate, C_r , for passive steel in concrete attacked by chlorides is about 100 $\mu\text{m}/\text{year}$. According to Mori and Ellingwood (1994), the typical corrosion rate, C_r , is a time-invariant random variable described by a lognormal distribution with mean C_r of 50 $\mu\text{m}/\text{year}$, and coefficient of variation V_{cr} of 50%. Because the corrosion rate changes with environment, no accurate data are available to predict the real corrosion rate.

3.7 EFFECT OF AGE ON CONCRETE STRENGTH

Another value affects concrete structure reliability: the strength of concrete. The concrete member capacity is usually a function of concrete cross-section dimensions, the steel bar area, the concrete compressive strength, and the steel reinforcement yield strength. After some years, the concrete element will deteriorate due to corrosion of the steel bars, which will reduce the steel cross-section dimension discussed in the previous sections. The yield strength will be the same with time, but the concrete cross-section dimension will be less effective due to concrete cracks and high reduction on the section in cases of falling concrete cover. The time required until spall of concrete cover will be discussed later.

There is something to gain: the concrete strength. Much research has discussed this in detail based on environmental conditions, which need to be addressed in detail in order to have a sense about reinforced concrete structure's reliability. In practice, one can see a deteriorated structure with corrosion on the steel bars without complete failure. With time, the concrete strength is increased; this increase in strength compensates for some of the member strength due to reduction in the steel cross-section area. One should not be satisfied with making a concrete design because of knowledge that the steel bars are carrying more load; any reduction on it will be very risky, so one should not depend on gaining strength on concrete with time. However, it does need to be taken into consideration.

Much research has been done to predict concrete strength after 28 days. In the majority of cases, the tests are conducted at concrete age of 28 days, when its strength is considerably lower than its long-term strength will be. Different methods have been suggested to predict concrete strength with age and different codes have different recommendations for predicting this strength. For example, Baykof and Syglof (1976) compared the gain in strength of concrete specimens stored in wet and dry conditions. They found that, in dry conditions, after 1 year there is no increase in concrete strength, as shown in figure 3.4. On the other hand, the strength of specimens stored in a wet environment (at 15°C) is considerably increased (shown in fig. 3.4).

In Madison, Wisconsin, Washa and Wendt (1975) tested concrete specimens stored in special environmental conditions to predict the concrete strength with age and found a valuable result. The specimens were moist cured for 28 days before placement outdoors on leveled ground in an uncovered, open location. Thermocouple data indicate that the outdoor compressive cylinders were subjected to about 25 cycles of freezing and thawing each winter. The relative humidity normally varied from 65–100%, with an average of 75%. The annual precipitation, including snow-fall, was about 32 in. Air temperatures usually ranged between 25 and 90°F (32 and 35°C). The average compressive strength with time is shown in figure 3.5.

Washa and Wendt (1989) concluded that the compressive strength of concrete cylinders made with cement that had a relatively low C_2S content, had a high surface area, and was stored outdoors for 50 years generally increased as a logarithm of the age for about 10 years. After 10 years, the compressive strength decreased or remained essentially the same. MacGregor (1983) used the same study and formulated an equation of the relation between compressive strength and age; in the case of 28-day-specified compressive strength of concrete, this is equal to

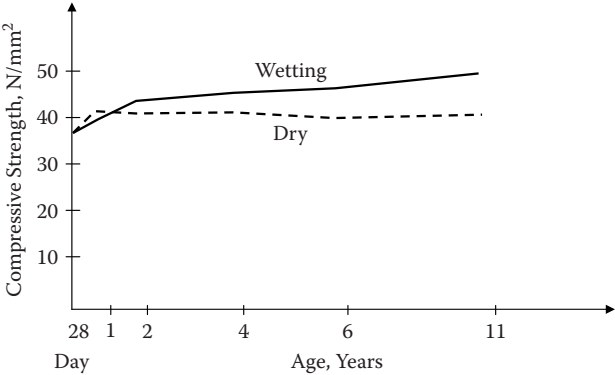


FIGURE 3.4 Variation of concrete strength with time.

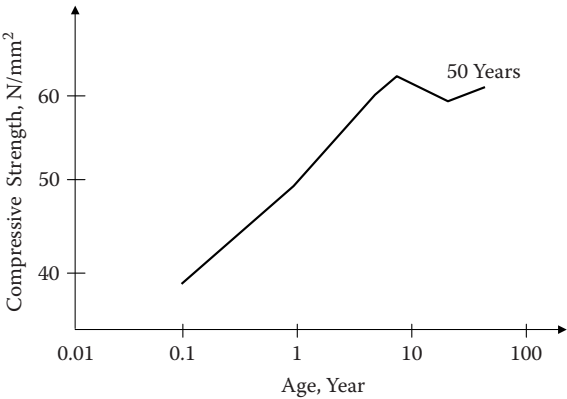


FIGURE 3.5 Variation of compressive strength with age. (Adapted from Washa, G. W., and K. F. Wendt. 1975. *ACI J. Proc.* 72(1):20–28.)

281.5 kg/cm² (27.6 Mpa). The corresponding mean of compressive strength at 28 days is 292.7 kg/cm² (28.7 Mpa):

$$f_c(t) = \begin{cases} 158.5 + 40.3 \ln(t) \dots\dots\dots \text{kg / cm}^2 \dots\dots\dots t < 10 \text{ years} \\ 489 \dots\dots\dots \text{kg / cm}^2 \dots\dots\dots t \geq 10 \text{ years} \end{cases} \tag{3.8}$$

where $f_c(t)$ is the concrete compressive strength with time and t is the time in days.

3.7.1 STATISTICAL ANALYSIS OF LONGTIME CONCRETE STRENGTH

The effect of concrete maturity, based on a number of studies of the longtime strength gain of concrete representative of lower bound relationships between age and strength of various classes of concrete, was obtained. These tended to show a linear relationship between strength and the logarithm of age. For lower strength concrete, 25 years' strength approaching 240% of the 28-day strengths was observed by Washa and Wendt in 1975. For high-strength concrete, the strength of old concrete approached 125–150% of the 28-day strength.

3.7.2 CODE RECOMMENDATIONS

Different codes recommend different methods to predict concrete strength with different age. In the following sections, the gain in concrete strength with age in the Egyptian, British, and Indian codes is discussed. Concrete compressive strength varies with age for normal concrete at moderate temperatures (Hilal 1987). The ratio of the concrete compressive strength at 28 days to that at a given concrete age may be estimated for normally and rapidly hardening portland cement according to Egyptian code (EC), as shown in the values given in table 3.2.

According to Nevil (1983), in the past, the gain in strength beyond the age of 28 days was regarded merely as contributing to an increase in the factor of safety of the structure. Since 1957 the codes of practice for reinforced and prestressed concrete allow the gain in strength to be taken into account in the design of structures that will not be subjected to load until an older age, except when no-fines concrete (concrete with little or no fine aggregate) is used; with some lightweight aggregates, verifying tests are advisable. The values of strength given in the British code of practice CP110:1972, based on the 28-day compressive strength, are given in table 3.3, but they do not, of

TABLE 3.2
Ratio of Fc28 to That at Age in Egyptian Code

Age (days)		3	7	28	90	360
Type of portland cement	Normal hardening	2.5	1.5	1.00	0.85	0.75
	Rapid hardening	1.80	1.30	1.00	0.90	0.85

Source: General Organization for Housing, Building, and Planning Research (GOHBPR). 1989. Egyptian Code of Practice for Design of Reinforced Concrete Structures. Cairo, Egypt.

TABLE 3.3
British Code of Practice CP110:1972 Factors of Increase
in Compressive Strength of Concrete with Age

Months	Age factor for concrete with a 28-day strength (Mpa)		
	20–30	40–50	60
1	1.00	1.00	1.00
2	1.10	1.09	1.07
3	1.16	1.12	1.09
6	1.20	1.17	1.13
12	1.24	1.23	1.17

course, apply when accelerators are used. This table shows that concrete continuously gains strength with time. The additional strength is about 20–25% of the corresponding 28-day strength. Most of the additional strength is gained within the first year.

3.7.3 AVAILABLE STATISTICAL PARAMETERS FOR CONCRETE STRENGTH CONSIDERING AGE

The available test results presenting the gain in concrete strength with age are discussed by Washa and Wendt (1984), who tested concrete specimens at 1, 5, 10, 25, and 50 years of age. The analysis of these data is shown in table 3.4, where statistical parameters for the ratio of concrete strength at 1, 5, 10, 25, and 50 years and at 28 days are calculated. Baykof and Sigalof (1976) discussed the gain in concrete strength of concrete specimens stored in wet and dry condition over time. The factor of increase in concrete strength with age at wet and dry conditions is presented in table 3.5. In general, buildings are always exposed to dry conditions after construction. Therefore, the case of dry conditions was taken into consideration in the study performed by El-Reedy, Ahmed, and Khalil (2000), which considered the factor of increased strength after 1 year by 1.25 to the strength at 28 days with coefficient of variation equal to 0.1. This gain remained constant throughout the lifetime of the building.

3.8 CORROSION’S EFFECT ON SPALLING OF CONCRETE COVER

Most problems that occur because of corrosion of steel in concrete are due not only to the shortage in the steel section but also to fall of the concrete cover. Many studies and much research have been conducted to calculate the amount of corrosion occurring and causing the concrete cover to fall. It has been found that cracks may occur in cases of reduction of 0.1 mm from steel reinforcement sections and, in some cases, much less than 0.1 mm, depending on the distribution of oxides and the ability of concrete to withstand the stresses, as well as the distribution of steel.

Various efforts have been made to estimate the amount of corrosion that will cause spalling of concrete cover. According to Broomfield (1997), the cracking is induced by less than 0.1 mm of steel bar section loss, although in some cases, far less than 0.1 mm has been needed. This is a function of the way that the oxide is

TABLE 3.4
Analysis of Results of Test for Gain of Concrete Strength with Time

Test no.	28 Days		1 Year		5 Years		10 Years		50 Years	
	Fc28	N/mm ²	Fc1	Fc1/Fc28	Fc5	Fc5/Fc28	Fc10	Fc10/Fc28	Fc50	Fc50/Fc28
1	28.1		36.5	1.30	52.8	1.88	55.3	1.97	53.8	1.92
2	35.4		45.9	1.30	61.7	1.75	61.2	1.73	64.8	1.83
3	30.1		40.8	1.35	52.0	1.73	55.2	1.83	53.5	1.78
4	31.8		42.3	1.33	53.0	1.67	56.4	1.77	55.7	1.75
5	43.5		57.6	1.32	63.4	1.46	73.6	1.69	73.1	1.68
6	35.1		43.9	1.25	54.9	1.57	55.4	1.58	56.5	1.61
7	36.2		51.9	1.44	55.8	1.54	59.9	1.66	62.0	1.71
8	44.1		58.4	1.33	73.3	1.66	71.3	1.62	72.4	1.64
9	36.1		51.5	1.43	62.3	1.73	63.4	1.76	55.9	1.55
10	36.2		47.2	1.31	53.7	1.48	55.6	1.54	50.2	1.39
11	49.6		61.1	1.23	70.3	1.42	66.9	1.35	65.0	1.31
12	38.5		51.8	1.34	59.1	1.53	57.5	1.49	50.6	1.31
Mean				1.33		1.62		1.67		1.62
St. dev. ^a				0.06		0.14		0.17		0.20
C.O.V. ^b				0.05		0.09		0.10		0.12

Source: Washa, G. W., and K. F. Wendt. 1975. *ACI Journal Proceedings* 72(1):20–28.

^a Standard deviation.

^b Coefficient of variation: equal to standard deviation divided by the mean.

TABLE 3.5
Factors of Increase in Concrete Strength with Age
in Dry and Wet Conditions

Age	Environment	
	Dry	Wet
1 Month	1.00	1.00
1 Year	1.25	1.20
2 Years	1.25	1.30
4 Years	1.25	1.40
6 Years	1.25	1.50
10 Years	1.25	1.63

Source: After Baykof, F., and Y. Sigalof. 1984. *Reinforced concrete structure*. Moscow: Mier.

distributed (i.e., how efficiently it stresses the concrete); the ability of the concrete to accommodate the stresses (by creep, plastic, or elastic deformation); and the geometry of bar distribution, which may encourage crack propagation by concentrating stresses, as in the case of a closely spaced series of bars near the surface or at a corner where there is less confinement of the concrete to restrain cracking.

From the corrosion rate measurements, it would appear that about 10-μm section loss or 30-μm corrosion growth is sufficient to cause cracking. However, corrosion is a complex mixture of oxides, and hydroxides and hydrated oxides of steel have a volume ranging from twice to about six times that of the steel consumed to produce it. According to El-Abiary et al. (1992), the time, t_s , in years between initiation of corrosion and spalling of concrete is calculated from the following equation:

$$t_s = \frac{0.08.C}{d.C_r} \tag{3.9}$$

where

- C = the concrete cover in millimeters
- d = the diameter of the steel bar
- C_r = the mean corrosion rate

Few structures collapse due to corrosion in steel reinforcement because the corrosion gives some warning and deterioration is evident from the color of the concrete and the presence of the cracks. This helps in making the appropriate decision to conduct the repair process in a timely manner, as is evident in some structures in figures 3.6, 3.7, and 3.8.

In figure 3.6, we clearly see the corrosion of steel stirrups: an almost completely ineffective resistance sector followed by concrete and steel corrosion—particularly, a steel bar in the corner where the corner is the fastest exhibition of the impact of carbonation and cracks on the concrete cover. Figure 3.7 shows corrosion of steel in the retaining wall of a bridge. Broomfield (1997) stated that a multistory garage



FIGURE 3.6 Photo of corrosion in concrete.



FIGURE 3.7 Photo of corroded abutment bridge wall.

building fell because of corrosion. Prestress bridges have also been found in England and Wales; the failure happened due to chlorides (also a bridge in Belgium, as stated by Woodward and Williams, [1988]). The spall of concrete cover for different structure members is clearly obvious in figures 3.7 and 3.8.

Major problems occur when there is black corrosion, particularly for structures used by the previous prestressing, as the corrosion does not form on the outer surface of steel. Often, the steel has been loaded to carry 50% or more of maximum tension resistance, and the lack of a steel reinforcement section due to corrosion strongly increases the likelihood of collapse.

Most problems, as we have said before, result from the fall of the concrete cover. For example, in New York City, a man was killed when a concrete slab bridge separated because the steel became corroded due to the use of salt to melt ice on the bridge. In Michigan, a similar accident occurred. Hence, we find that the corrosion of steel must be under control so as not to lead to fall of the concrete cover. This fall results from oxides generated on the surface of steel, which is 10 times the volume of its structural size, and the fact that steel buried in the concrete generates strong



FIGURE 3.8 Photo of cracks in columns of elevated water tanks.



FIGURE 3.9 Photo of cracks on columns and beams.

increases in volume. This would affect the weakest side (the concrete cover), which is where cracks form and then cause the concrete to fall.

The process of corrosion formation on steel was shown in figure 3.8. Also, cracks form and the concrete cover falls, as in figure 3.9. Note that the concrete cover in the corner is more prone to falling because it is a largely exposed area for the penetration of carbon dioxide or exposure to chlorides, as well as oxygen. Therefore, concrete cracks often happen faster in this situation.

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4 Assessment Methods for Reinforced Concrete Structures

4.1 INTRODUCTION

In chapter 2, the corrosion of steel buried in concrete was studied and it was found that the corrosion has the same characteristics in terms of its impact on steel in any situation. The causes of corrosion or its types that affect the steel have been studied in detail in chapter 3. When we see that a concrete structure has signs of corrosion, we want to repair it, but before we do that, we must evaluate the building with high accuracy and identify the environmental conditions surrounding it.

After performing risk assessment on the structure, now we can identify the parts of the concrete elements that will be repaired and determine the method of repair. Whenever the assessment method is correct and technically accurate to determine the degree of building risk, the repair process will be technically accurate and precise. The process of building assessment is the preliminary step; it diagnoses the defects in a structure as a result of corrosion and identifies the causes that led to the corrosion. Therefore, we will think in two directions: one to determine the method of repair and the other to define a reasonable method to protect steel bars from corrosion in the future.

The process of assessing structures often takes place in two stages. The first stage involves the initial assessment of the building, definition of the problem, and development of the plan for detailed evaluation of the structure. A detailed assessment of the building will carefully define the problems and the reasons for them; at this stage, the detailed inspection of the whole building will be carried out. A basis has been established for the inspection of a building to evaluate it through technical report 26 (Concrete Society 1984), as well as through American Concrete Institute (ACI) standards.

There are many reasons for the deterioration of reinforced concrete structures, from the presence of cracks to falling of the concrete cover. Among those reasons are increases in stresses, plastic shrinkage, frost or cracked concrete for plastic concrete, moving the wood form during implementation of construction, or the presence of aggregate, which has the capability to shrink or deteriorate as a result of the interaction of the aggregate with the alkalinity. There are also some structures, such as reinforced concrete pipelines in sanitary projects and underground concrete structures like tunnels, with surfaces in direct contact with the soil. In these cases, the carbonate present in the water attacks the concrete. Although this type of deterioration is

important to consider, we will focus here on corrosion as a result of concrete structures' exposure to the atmosphere.

As we have stated previously, there are several types of cracks. Therefore, the engineer responsible for performing the evaluation must be highly experienced in that area so that he can specify precisely why the deterioration in concrete occurs. The wrong diagnosis will result in the wrong repair, which will cause heavy financial losses and have an impact on the safety of the structure.

The evaluation of a structure is not only a structural assessment, but also an assessment of the state of concrete in terms of the presence of corrosion and the rate of corrosion and collapse of concrete. From previous information, we will have the ability to decide if the concrete member can withstand loads. In addition, a case study will be done to assess the member to ascertain whether a deflection more than the allowable limit will cause cracks, reduction in cross-section area, or falling concrete cover.

4.2 PRELIMINARY INSPECTION

In this inspection, it is necessary to identify clearly the geographical location of the building, the nature and circumstances of weather conditions surrounding the building, the method of constructing the building, its structural system, and the method of loading. This assessment is often performed by a preliminary visual inspection concentrated on cracks and the fall of concrete. It also focuses on collecting data about the thickness of the concrete cover and the nature of the structure in terms of the quality of concrete and of construction, as well as the structure's system—whether it consists of a beam and column system, or slab on load-bearing masonry, or prestressed or precast concrete. It is necessary to perform some simple measurements—for example, to determine the depth of the transformation of carbon in the concrete. A sample can be taken from concrete that has fallen and laboratory tests performed.

The safety of the structure must be calculated precisely, especially after reducing the cross-sectional area of steel due to corrosion; the fall of the concrete cover reduces the total area of the concrete member. These two areas are the main coefficients that directly affect the capacity of the concrete member to carry the loads.

4.3 DETAILED INSPECTION

The purpose of the detailed inspection is to determine accurately whenever possible the degree of seriousness and the deterioration of the concrete. Therefore, we need to know the amount of the collapse that has occurred, the cause of the deterioration in the concrete, and the amount of repair that will be needed. These must be defined precisely at this stage, as such quantities will be put forward to contractors for repair. At this stage we will need to be familiar with the reasons for the collapse in detail and the capability of performing the failure analysis technique.

Initially, visual inspection may be in conjunction with the use of a small hammer and going to other measurements to determine the depth of carbon transformation in the concrete, as well as the degree of steel corrosion in concrete and how much it extends in the steel bars. Moreover, in this stage it is important to define the

degree of concrete electric resistivity to predict the corrosion rate. All these required measurements will be discussed in this chapter. As you should know, the weather conditions affecting the building are the main factor that affects the measurement readings; they also affect the selection of the method of repair.

4.4 METHODS OF STRUCTURE ASSESSMENT

There are several methods to assess the structure in terms of the degree of extension of corrosion in steel reinforcement bars and its impact on the whole structure. The first and most important method is the visual inspection because it is not expensive and easily performed. This is followed by other methods that require certain capabilities and are often used in the case of structures of special importance that need expensive repair. Therefore, the use of sophisticated, highly accurate technology is required to identify the degree of corrosion of the steel reinforcement in concrete because this affects the total cost of the structure—for example, concrete bridges or special buildings such as parking garages or tunnels.

Each of the measurement tools used has a specific accuracy with advantages and disadvantages. In general, the process of assessing the structure for corrosion attack must be conducted by a person who has an acceptable amount of experience in evaluating the process of corrosion because experience is the master key and plays a big role in successful assessment. However, some devices need specialized knowledge for their use, as well as experienced knowledge of the accuracy and environmental factors that affect the equipment readings and how to overcome them.

In table 4.1 every method is identified, as well as the user’s ability to work with that method and the performance rate. This might help to estimate the cost of the inspection used to evaluate the building; the performance rates will assist us in performing the inspection on schedule. These measurements will determine the cause of corrosion and the degree of passive protection layer for the steel bars, as well as the corrosion rate expected in every part of the building. From this information, we can

TABLE 4.1
Practical Methods to Evaluate Concrete Structures

Methods	Inspection	User	Approx. performance rate
Visual inspection	Surface defects	General	1 m ² /sec
Chain or hammers	Void behind cover	General	0.1 m ² /sec
Concrete cover measurement	Distance between steel bars and concrete surface	General	One reading every 5 min
Phenolphthalein	Carbonation depth	General	One reading every 5 min
Half cell	Evaluate corrosion risk	Expert	One reading/5 sec
Linear polarization	Corrosion rate	Expert	One reading in 10–30 min
Radar	Defects and steel location	Expert	1 m/sec by using car or 1 m ² in 20 sec ^a

^a Add more time than that during schedule plan preparation.

determine what kind of repair the structure needs and its type of construction, calculate the quantity, and survey the repair required to strengthen the concrete member.

4.4.1 VISUAL INSPECTION

Visual inspection is the first step in any process of technical diagnosis and generally starts by viewing the structure as a whole and then concentrating on the general defects. After that, it is necessary to define precisely the corrosion deterioration and the extent of corrosion on the steel reinforcement. As we have stated before, the process of assessing the building must be performed by an expert because the cracks in the concrete structure may not be the cause of corrosion; corrosion is not the only factor that causes cracks. However, it is the main factor for major deterioration of structures. Later, figures 4.2a, 4.2b, and 4.2c will illustrate the various forms and causes of cracks in beams, slabs, and columns.

In general, cracks in concrete have many causes. They may affect appearance only, or they may indicate significant structural distress or a lack of durability. Cracks may represent the total extent of the damage, or they may point to problems of greater magnitude. Their significance depends on the type of structure, as well as the nature of the cracking. For example, cracks that are acceptable for building structures may not be acceptable in water-retaining wall structures.

The proper repair of cracks depends upon knowing the causes of the cracks and selecting repair procedures that take these causes into account; otherwise, the repair may only be temporary. Successful long-term repair procedures must prevent the causes of the cracks as well as the cracks themselves. The cracks may occur in plastic concrete or in hardened concrete. Plastic concrete cracks occur as plastic shrinkage cracking, settlement cracking, and, after hardening, dry shrinkage cracking. The following sections will illustrate these types of cracking based on ACI code.

4.4.1.1 Plastic Shrinkage

Cracking caused by plastic shrinkage in concrete occurs most commonly on the exposed surfaces of freshly placed floors and slabs or other elements with large surface areas when they are subjected to a very rapid loss of moisture caused by low humidity and wind or high temperature or both. Plastic shrinkage usually occurs prior to final finishing, before curing starts. When moisture evaporates from the surface of freshly placed concrete faster than it is placed by curing water, the surface concrete shrinks. Due to the restraint provided by the concrete on the drying surface layer, tensile stresses develop in the weak, stiffening plastic concrete, resulting in shallow cracks that are usually not short and run in all directions. In most cases, these cracks are wide at the surface. They range from a few millimeters to many meters in length and are spaced from a few centimeters to as much as 3 m apart. Plastic shrinkage cracks may extend the full depth of elevated structural slabs.

Since cracking because of plastic shrinkage is due to a differential volume change in the plastic concrete, successful control measures require a reduction in the relative volume change between the surface and other portions of the concrete. There are many methods and techniques to prevent this type of crack in case of rapid loss of moisture due to hot weather and dry winds. These methods include the use of fog

nozzles to saturate the air above the surface and using plastic sheeting to cover the surface between the final finishing operations. In many cases, during construction it is preferable to use wind breakers to reduce the wind velocity; sunshades to reduce the surface temperature are also helpful. Additionally, it is good practice to schedule flat work after the walls have been erected.

4.4.1.2 Settlement

After initial placement, vibration, and finishing, concrete has a tendency to continue to consolidate. During this period, plastic concrete may be locally restrained by reinforcing steel, a prior concrete placement, or formwork. This local restraint may result in voids and/or cracks adjacent to the restraining element, as shown in figure 4.1. When associated with reinforcing steel, settlement cracking increases with increasing bar size, increasing slump, and decreasing cover. The degree of settlement cracking will be magnified by insufficient vibration or the use of leaking or highly flexible forms. Proper form design and adequate vibration, provision of sufficient time intervals between the placement of concrete in slabs and beams, the use of the lowest possible slump, and an increase in concrete cover will reduce settlement cracking.

4.4.1.3 Shrinkage from Drying

The common cause of cracking in concrete is shrinkage due to drying. This type of shrinkage is caused by the loss of moisture from the cement paste constituent, which can shrink by as much as 1% per unit length. Unfortunately, aggregate provides internal restraint that reduces the magnitude of this volume change to about 0.05%. Upon wetting, concrete tends to expand.

These moisture-induced volume changes are a characteristic of concrete. If the shrinkage of concrete could take place without any restraint, the concrete would not crack. It is the combination of shrinkage and restraint, which is usually provided by another part of the structure or by the subgrade, that causes tensile stresses to develop. When the tensile stresses of concrete are exceeded, it will crack. Cracks may propagate at much lower stresses than are required to cause crack initiation.

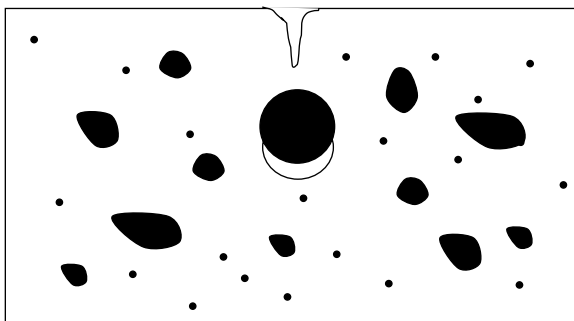


FIGURE 4.1 Settlement cracking.

In massive concrete structure elements, tensile stresses are caused by differential shrinkage between the surface and the interior concrete. The larger shrinkage at the surface causes cracks to develop that may, with time, penetrate more deeply into the concrete. Magnitude of the tensile stresses is influenced by a combination of factors:

- amount of shrinkage
- degree of restraint
- modulus of elasticity
- amount of creep

The amount of drying shrinkage is influenced mainly by the amount and type of aggregate and the water content of the mix. The greater the amount of aggregate is, the smaller is the amount of shrinkage. The higher the stiffness of the aggregate is, the more effective it is in reducing the shrinkage of the concrete. This means that concrete containing sandstone aggregate has a higher shrinkage rate—about twice that of concrete containing granite, basalt, or limestone. The higher the water content is, the greater is the amount of shrinkage from drying.

Surface crazing on walls and slabs is an excellent example of shrinkage from drying on a small scale. Crazing usually occurs when the surface layer of the concrete has higher water content than that of the interior concrete. The result is a series of shallow, closely spaced, fine cracks. Drying shrinkage can be reduced by using the maximum practical amount of aggregate in the mix. The lowest water-to-cement ratio is important to avoid this type of shrinkage. A procedure that will help reduce settlement cracking, as well as drying shrinkage in walls, is to reduce the water content in concrete as the wall is placed from the bottom to the top. Using this procedure, bleed water from the lower portions of the wall will tend to equalize the water content within the wall. To be successful, this procedure needs careful control and proper consolidation.

Cracking due to shrinkage can be controlled by using properly spaced contraction joints and proper steel detailing. It may also be controlled by using shrinkage-compensating cement.

4.4.1.4 Thermal Stresses

The temperature differences within a concrete structure may be due to cement hydration or changes in ambient temperature conditions or both. These temperature differences result in differential volume changes. The concrete will crack when the tensile strains due to the differential volume changes exceed their tensile strain capacity.

The effects of temperature differentials due to the hydration of cement are normally associated with massive concrete forms such as large columns, piers, beams, footing, retaining walls, and dams, while temperature differentials due to changes in the ambient temperature can affect any structure. Considering thermal cracking in massive concrete forms, production procedures for portland cement heat it as

it hydrates, causing the internal temperature of concrete to rise during the initial curing period. The concrete rapidly gains both strength and stiffness as cooling begins. Any restraint of the free contraction during cooling will result in tensile stress. Tensile stresses developed during the cooling stage are proportional to the temperature change, the coefficient of thermal expansion, the effective modulus of elasticity, and the degree of restraint. The more massive the structure is, the greater is the potential for temperature differential and degree of restraint.

Procedures to help reduce thermally induced cracking include reducing the maximum internal temperature, delaying the onset of cooling, controlling the rate at which the concrete cools, and increasing the tensile strain capacity of the concrete.

Hardened concrete has a coefficient of thermal expansion that may range from 7 to $11 \times 10^{-6}/^{\circ}\text{C}$, with an average of $10 \times 10^{-6}/^{\circ}\text{C}$. When one portion of a structure is subjected to a temperature-induced volume change, the potential for thermally induced cracking exists. Designers must give special consideration to structures in which some portions are exposed to temperature changes while other portions of the structure are either partially or completely protected. A drop in temperature may result in cracking in the exposed element, while increases in temperature may cause cracking in the protected portion of the structure. Therefore, the designer must allow movement of the structure by recommending using contraction joints and providing the correct detailing to it.

Structures that experience high differences in temperature are usually concrete structures built in areas near a desert, where temperatures can vary greatly between afternoon and midnight. Moreover, countries with high temperatures usually use air-conditioning inside buildings, so it will be a high probability to have cracks due to the difference in temperatures inside and outside a building. The designer should take these stresses into consideration.

4.4.1.5 Chemical Reaction

As shown in figure 4.2a, cracks like a star in the concrete surface are an indication of a chemical reaction. This reaction occurs when an aggregate is present that contains active silica and alkalis derived from cement hydration, admixtures, or external sources such as curing water, ground water, or alkaline solutions stored or used in the finished structure.

The alkali-silica reaction results in the formation of a swelling gel, which tends to draw water from other portions of the concrete. This causes local expansion and accompanying tensile stresses, and may eventually result in the complete deterioration of the structure.

Ground water that has sulfate is a special durability problem for concrete. The sulfate penetrates hydrated cement paste so that it comes in contact with hydrated calcium aluminates. Calcium sulfoaluminate will be formed, accompanied by increase in the volume, which will result in high tensile stresses that cause cracking. Therefore, using portland cement types II and V is recommended because they contain low tricalcium aluminates that will reduce the severity of the cracks.

As noted in figures 4.2a, 4.2b, and 4.2c, in cases of corrosion in the steel reinforcement, the cracks will be parallel to the steel bars for concrete members such as beams, slabs, and columns. Moreover, spots of brown color on the concrete surface are an indication of the corrosion in the steel reinforcement in this stage. Only experienced engineers should perform the inspection—using photos; recording remarks in a diary, noting the weather, date, and time; and observing the mode of execution and quality of concrete. These spots and the emergence of infrastructure can be seen on the beam on the bridge as well as on the column in figure 4.3.

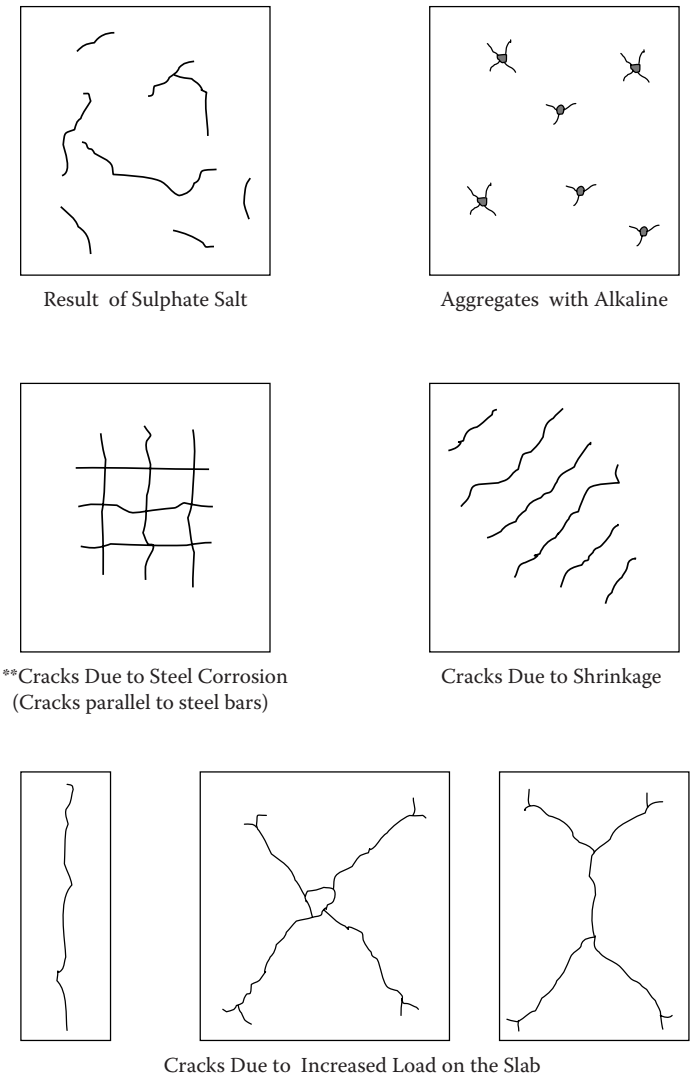


FIGURE 4.2a Types of cracks in reinforced concrete slabs.

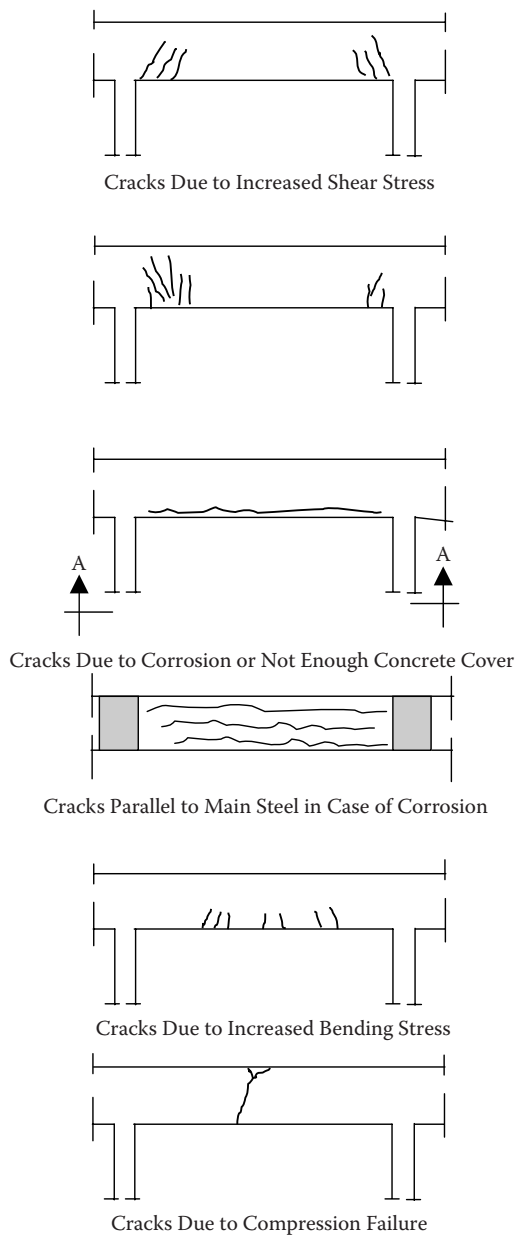


FIGURE 4.2b Types of cracks in reinforced concrete beams.

Some bodies, such as the Strategic Highway Research Program (SHRP), have a fixed system that the inexperienced engineer can follow through some conditions and limitations; it can define the defects precisely as well as the cause of such defects.

The disadvantage of visual inspection is that it depends completely on the experience of the engineer performing the inspection because some cracks have more

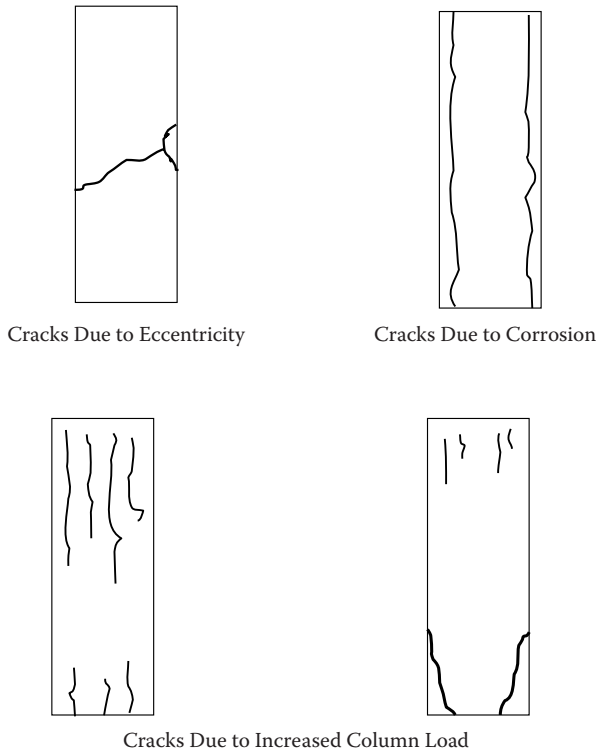


FIGURE 4.2c Types of cracks in reinforced concrete columns.

than one cause, with or without the corrosion effect. The following data must be collected before performing the visual inspection and must be stated in the structure assessment report:

- construction year
- contractor and engineering office data
- structural system
- drawings and project specifications
- construction method
- concrete tests, if available
- environmental condition (near sea or chemical factory)

The owner must provide these data to the engineer, if he is a third-party expert. In most cases, all these data cannot be collected because, logically speaking, a building may be over 20 or 30 years old; therefore, the drawings and specifications will not be available because computers and software drawing programs were not common 20 years ago. All the hard-copy drawings, in most cases, are not available or in bad condition. If the owner has a big organization with an engineering department, the engineers may have a document control system and the consulting engineer can



FIGURE 4.3 Brown spots on bridge column and girder.

find the drawing. This can be possible for office and industrial buildings, but for residential building, the engineer will be very lucky if the owner has these drawings.

The construction year is very critical information because the codes and specifications that were applied in this year can be defined from this, although this depends on the engineer's experience. Knowing the construction year, the contractor who constructed the building, and the engineering office that provided the engineering design, the engineer can imagine the condition of the building based on the reputation of the contractor and the engineering office.

The construction system can be ascertained by visual inspection, the drawing, or talking with the user of the building and the engineering firm that shared in reviewing the engineering design documents or supervising the construction.

Information on environmental conditions will be provided by the owner. The engineer can see it on-site, collect the data about the environmental condition from other sources, and take into consideration all the data, including temperatures in summer and winter, morning and night, and the wind and wave effects.

An owner must be sure that all the data are delivered to the consultant who does the inspection because any data are valuable to him. He needs to remember "garbage in, garbage out" and verify all the data.

In industrial facilities, plant conditions and mode of operations are not clear to the consultant engineer, so this information must be delivered to him. Machine vibrations, heat of liquid in pipelines, and so on have more impact on the concrete

structure assessment result. When data are collected from a processing plant or factory, it is necessary to consider that the mode of operations may change from time to time. Therefore, all these histories must be delivered to the engineer conducting the evaluation, who is usually a structural engineer unfamiliar with and unable to imagine these situations.

If the data of wind and wave do not exist or are not known, they must be obtained from a third party specializing in meteorological or oceanological information. These data are very important in cases of offshore structures or marine structures. Their assessment depends on these data if wind and wave represent the greatest loads affecting the structure.

The concrete compressive strength test is usually performed in any building by cylinder specimens, as in American code, or cube specimens, as in the British standard. However, the problem is to find the data after 20 years or more, depending on the age of the building. If they are not available, the test must be performed using an ultrasonic pulse velocity test, Schmidt hammer, or core test.

4.4.2 CONCRETE TEST DATA

The following sections describe tests that should be performed to obtain data on concrete strength if these data are not available. There are variations among these tests concerning accuracy, effects on the building, and cost. Therefore, the consulting engineer has the responsibility to decide whether any of these tests is suitable to the building to be evaluated. Moreover, there are some precautions for these tests: When the samples are taken, they must be based on the visual inspection results that were done before. Also, when samples are taken or the loading test is conducted, it must be accompanied by a suitable temporary support to the structure and the adjacent members. These temporary support locations and strengths should be suitable to withstand the load in case of weakness in the structure member that is being tested. These tests will follow ACI 228-89 R1 (1989) and BS 1881 (1971a, 1971b, 1983).

4.4.2.1 Core Test

This test is considered one of the semidestructive tests. It is very important and popular for study of the safety of a structure as a result of changing the system of loading or a deterioration of structure as a result of accidents, such as fire, or weather factors. It is also used when temporary support is needed for repair and no accurate data about concrete strength are available. This test is not too expensive and is the most accurate test to determine the strength of concrete actually carried out.

The core test (American Concrete Society 1987; ASTM C 42-90m; BS 1881 1983) is done through cutting cylinders from the concrete member, which could affect the integrity of the structure. Therefore, the required samples must be taken according to the standard as the required number will provide adequate accuracy of the results without weakening the building.

In our case for a structure that has deteriorated due to corrosion of steel bars, the structure has lost most of its strength due to reduction on the steel cross-section area. Thus, more caution must be used when performing this test and selecting the proper concrete member on which to perform it so that there will be no effect on the building

TABLE 4.2
Number of Cores and Deviation in Strength

Number of cores	Deviation limit between expected strength and actual strength (confidence level 95%)
1	+12%
2	+6%
3	+4%
4	+3%

from a structural point of view. Therefore, the codes and specifications provide some guides to the number of cores to test; these values are:

- Volume of concrete member (V) ≤ 150 m³ takes 3 cores.
- Volume of concrete member (V) > 150 m³ takes 3 + (V – 150/50) cores.

The degree of confidence of the core test depends on the number of tests, which must be the minimum possible. The relation between number of cores and confidence is found in table 4.2.

Before the consulting engineer chooses the location of the sample, he must define the location of the steel bars to assist in selecting the location of the sample away from them in order to avoid taking samples containing steel reinforcement bars, if possible. He must carefully determine the places to preserve the integrity of the structure; therefore, this test should be performed by an experienced engineer who conducts such an experiment taking precautions, determining the responsibility of individuals, and accurately reviewing the nondestructive testing that had been conducted. Figures 4.4 and 4.5 present the process of taking the core from a reinforced concrete bridge girder.

Core size. Note that the permitted diameter is 100 mm in the case of maximum aggregate size of 25 mm and 150 mm in the case of maximum aggregate size not exceeding 40 mm. It is preferable to use a 150-mm diameter whenever possible, as it gives more accurate results. This is shown in table 4.3, which represents the relationship between the dimensions of the sample and potential problems. This table should be considered when choosing a reasonable core size. Some researchers have stated that the core test can be done with a core diameter of 50 mm in the case of maximum aggregate size and not more than 20 mm overall; note that small core sizes give results different from those of large sizes.

Because of the seriousness of the test and inability to take high numbers of samples, gathering the sample should be well supervised. Moreover, the laboratory test must be certified and the test equipment must be calibrated and a certificate of calibration from a certified company must be held.

Sample extraction uses pieces of a cylinder, that differ depending on the country. Cylinders are equipped with a special alloy mixture with diamond powder to feature pieces in the concrete during the rotation of the cylinder through the body.



FIGURE 4.4 Taking a core sample.



FIGURE 4.5 How to take a core sample.

TABLE 4.3
Core Size with Possible Problem

Test	Diameter (mm)	Length (mm)	Possible problem
1	150	150	May contain steel reinforcement
2	150	300	May cause more cutting depth to concrete member
3	100	100	Not allowed if maximum aggregate size is 25 mm
			May cut with depth less than required
4	100	200	Less accurate data

As a precaution, sampling methods should match and bring appropriately consistent pressure to bear; this depends on the expertise of the technician.

After that, the core will be filled by dry concrete with suitable strength or grouting will be poured; the latter is a popular method. Another solution depends on using epoxy and injecting it into the hole and then inserting a concrete core of the same

size to close the hole. No matter which method is chosen, the filling must be done soon after the cutting. The filling material will be standing by the technician who does the cutting, as this core is affecting the integrity of the structure.

The lab must examine and photograph every core and note gaps identified within the core as small voids, if they measure between 0.5 and 3 mm; voids, if they measure between 3 and 6 mm; or big voids, if they measure more than 6 mm. The lab also examines whether the core is of nesting and determines the shape, kind, and color gradient of aggregates, as well as any apparent qualities of the sand. In the laboratory, dimensions, weight of each core, density, steel bar diameter, and distance between the bars will be measured.

Sample preparation for test. After the core is cut from the concrete element, the sample is then processed for testing by leveling the surface of the core. A core is taken that has a length of not less than 95% of the diameter and not more than double the diameter. Figure 4.6 shows the shape of the core sampling after cutting from the concrete structure member directly.

For the leveling surface, a chainsaw, spare concrete, or steel cutting disk is used. After that, the two ends of the sample are prepared by covering them with mortar or sulfide and submerging the sample in water at a temperature of $20 \pm 2^\circ\text{C}$ for at least 48 hours before testing it. The sample is put in a machine test and an influence load with the rate of regular and continuous range of $0.2\text{--}0.4\text{ N/mm}^2$ is applied gradually, until it reaches the maximum load at which the sample has been crushed.



FIGURE 4.6 Shape of core sample.

The estimated actual strength for a cube can be determined by knowing the crushing stress obtained from the test and using the following equation, as λ is the divided the core length by its diameter. In the case of a horizontal core, the strength calculation will be:

$$\text{Estimated actual strength for cube} = 2.5/(1/\lambda) + 1.5 \times \text{core strength}$$

where λ = core length/core diameter.

In the case of a vertical core, the strength calculation is:

$$\text{Estimated actual strength for cube} = 2.3/(1/\lambda) + 1.5 \times \text{core strength} \quad (4.1)$$

In the case of existing steel in the core perpendicular to the core axis, the previous equations will be multiplied by the following correction factor:

$$\text{Correction factor} = 1 + 1.5 (s \phi)/(LD) \quad (4.2)$$

where

L = the core length

D = core diameter

s = distance from steel bar to edge of core

ϕ = steel bar diameter

Cores are preferred to be free of steel; if steel is found, it is necessary to use the correction factor, taking into account that it is taken only in the event that the value ranges from 10–5%. We must agree to use the results to core, but if the correction factor is more than 10%, the core's results cannot be trusted and another core should be taken.

When examining the test results, certain points must be taken into account when evaluating the results. Before testing, the sample should be submerged in water; this leads to a decrease in strength to up to about 15% for the strength of dry concrete. The equation to calculate the expected actual concrete strength does not take into account any differences in direction between the core and the standard cube direction. It states that the concrete is acceptable if the average strength of the cores was at least 75% of the required strength and the calculated strength for any core was less than 65% of the required strength. In the case of prestressed concrete, the concrete strength is acceptable if the average strength of the cores is at least 80% of the required strength and the calculated strength for any core is less than 75% of the required strength.

4.4.2.2 Rebound Hammer

This is nondestructive testing, so it is useful in determining estimated concrete compressive strength. This is the most common test, as it is easy to do and very inexpensive compared with other tests; however, it gives less precise outcomes of data results. This test relies on measuring the concrete strength by measuring the hardening from the surface. It will be able to identify the concrete compressive strength of the concrete member by using calibration curves of the relationship between reading the concrete



FIGURE 4.7 Rebound hammer.



FIGURE 4.8 Type of rebound hammer.

hardening and concrete compressive strength. Figures 4.7, 4.8, and 4.9 present different types of rebound (Schmidt) hammer from ways of reading the results.

Rebound hammers most commonly give impact energy of 2.2 N/mm^2 . There is more than one way to show results, based on the manufacturer; in some cases, the reading will be an analog or digital number or connect to a memory device to record the readings. Before it is used, the rebound hammer should be inspected using the calibration tools that are with the device when it is purchased. The calibration should be within the allowable limit based on manufacturer recommendation.

The first and most important step in the test is to clean and smooth the concrete surface at the sites that will be tested by honing in an area of about $300 \times 300 \text{ mm}$. Preferably, the surface should be tested to determine that it has not changed after casting or that no smoothing took place during the casting process. On the surface to be tested, a net of perpendicular lines is drawn in both directions 2–5 cm apart. The intersection points will be the points to be tested; the test point must be about 2 cm from the edge. Figures 4.9 and 4.10 show that the surface must be cleaned before testing and that the rebound hammer is perpendicular to the surface.

The following recommendations should be followed during the test:

- The hammer must be perpendicular to the surface that will be tested in any conditions because the direction of the hammer affects the value of the rebound number as a result of the impact of hammer weight.

- The wet surface gives a significantly lower reading of the rebound hammer than a dry surface does—up to 20%.
- The tested concrete member must be fixed and not vibrate.
- The curves for the relationship between concrete compressive strength and rebound number as given from the manufacturer cannot be used; rather, the hammer must be calibrated by taking a reading on concrete cubes and crushing the concrete cubes to obtain the calibration of the curves. It is important to perform this calibration from time to time as the spring inside the rebound hammer loses some of its stiffness with time.

Only one hammer must be used when a comparison is made between the quality of concrete in different sites. Type of cement affects the readings, as in the case of concrete with high-alumina cement, which can yield higher results (about 100%) than those of concrete with ordinary portland cement. Concrete with sulphate-resistant cement can yield results about 50% lower than that using ordinary portland cement. Higher cement content gives a reading lower than that of concrete with lower cement content; in any case, the gross error is only 10%.



FIGURE 4.9 Doing a test.



FIGURE 4.10 Test and read the number.

Data analysis. The number of readings must be high enough to give reasonably accurate results. The minimum number of readings is 10, but usually 15 readings are taken. Extreme values will be excluded and the average taken for the remaining values. From this, the concrete compressive strength will be known.

4.4.2.3 Ultrasonic Pulse Velocity

This test is one of the nondestructive testing types (ACI 228-89-R1 1989; BS 1881 1971a, 1971b; Bungey 1993); its concept is to measure the speed of transmission of ultrasonic pulses through the construction member by measuring the time required for the transmission of impulses. When the distance between the sender and receiver is known, the pulse velocity can be calculated.

These velocities are calibrated by knowing the concrete strength and its mechanical characteristics. These can then be used for any other concrete by the same procedure to identify compressive strength, dynamic and static modulus of elasticity, and Poisson ratio. The equipment must have the capability to record time for the tracks with lengths ranging from 100–3000 mm accurately + 1%. The manufacturer should define how the equipment works in different temperatures and humidities. It must be available as a power transformer sender and the receiver of natural frequency vibrations between 20 and 150 kHz, bearing in mind that the frequency appropriate for most practical applications in the field of concrete is 50–60 kHz. There are different ways for wave transmission as surface transmission, as shown in figures 4.11a and 4.11b.

Semidirect transmission is clearly shown in figures 4.12a and 4.12b. The third type of wave transmission is a direct transmission, as shown in figures 4.13a and 4.13b. It is given with the UT (ultrasonic testing) equipment—two metal rods with lengths of 250 and 1000 mm. The first is used to determine the zero of the measurement and the second is used in the calibration. In both cases, each rod indicates the time of the passage of waves through it.

Hence, the ends of the rod are appropriately connected by sender and receiver. They measure the time for pulse transmission and compare it with the known reading; the small rod, if there are any deviations, adjusts the zero of the equipment to

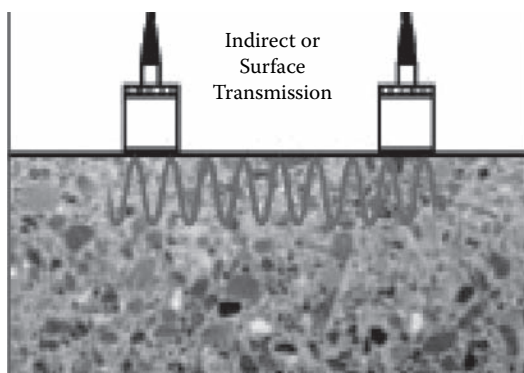


FIGURE 4.11a Surface transmission.



FIGURE 4.11b Surface transmission.

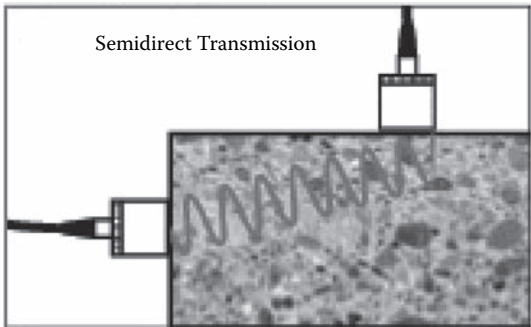


FIGURE 4.12a Semidirect transmission.



FIGURE 4.12b Semidirect transmission.

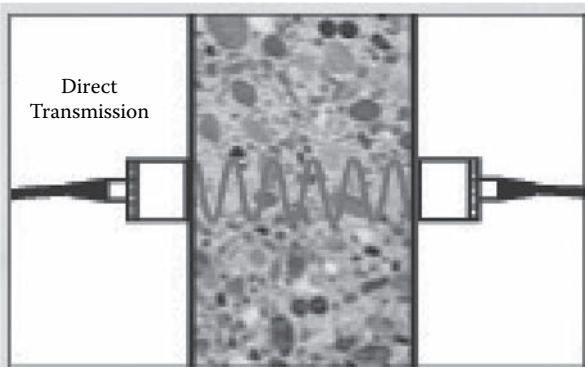


FIGURE 4.13a Direct transmission.



FIGURE 4.13b Direct transmission.

provide the known reading. A long bar is used in the same way to define accuracy of results; in this case, the difference between the two readings cannot be more than $\pm 0.5\%$ to qualify them as measurements with the required accuracy.

The wave transmission velocity value in steel is twice the value of that in concrete; thus, in the case of existing steel bars in a concrete member, doing a test on it will influence the accuracy of the reading as it will be a high value for the wave impulse velocity. To avoid this, the location of the steel reinforcement must be defined previously with respect to the path of the ultrasonic pulse velocity.

To correct the reading, one must consider the reading for steel bars parallel to the path of the pulse wave, as shown in figure 4.14. The calculation of the pulse

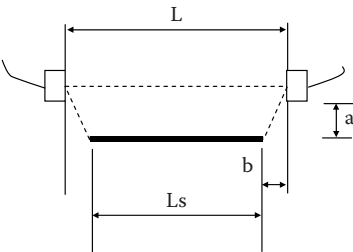


FIGURE 4.14 UT wave parallel to the steel bars.

velocity will be as in the following equation. Time of transmission will be calculated by the equipment and by ascertaining the wave path, the wave velocity in steel, and the distance between the two ends:

$$V_c = K.V_m \quad (4.3)$$

$$K = \gamma + 2(a/L)(1 - \gamma^2)^{0.5} \quad (4.4)$$

$$L_s = (L - 2b) \quad (4.5)$$

where

V_m = pulse velocity from transmission time from the equipment

V_c = pulse velocity in concrete

γ = a factor whose value varies according to steel bar diameter

The effect of the steel bar can be ignored if the diameter is 6 mm or less or if the distance between the steel bar and the end of the equipment is large.

In the case of steel reinforcement bars, the axis is perpendicular to the direction of pulse transmission, as shown in figure 4.15. Steel will have less effect on the reading in this case (figure 4.16). The effect can be considered zero if a transmission source of 54 kHz is used and the steel bar diameter is less than 20 mm. The preceding equation is used in the case of less frequency, a diameter greater than 20 mm, and by changing the value of γ according to the bar diameter by the data delivered by the equipment.

Other factors influence the measurement: Temperature, thawing, concrete humidity, and the factor effect must be considered. *Most common errors* include:

- Ignore using the reference bar to adjust zero and this will impact the accuracy of the results.
- The concrete surface is level. Smoothing it after pouring may result in properties different from the concrete in the core of the member and therefore such measurements should be avoided as much as possible. If avoidance is not possible, one must take into account the impact of the surface.
- Temperature affects the transmission of ultrasonic velocity. According to table 4.4, this fact must be taken into account for an increase or decrease in temperature of 30°C.
- When comparing the quality of concrete between the various components of the same structure, similar circumstances should be taken into account in terms of the composition of concrete, moisture content and age, temperature, and type of equipment used. There is a relationship between the quality of concrete and the speed, as in table 4.5.

The static and dynamic modulus of elasticity can be defined by knowing the transmission pulse velocity in the concrete, as shown in table 4.6.

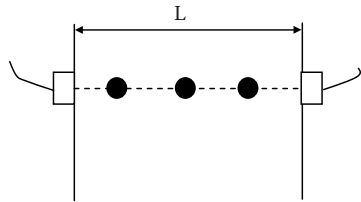


FIGURE 4.15 UT wave perpendicular to the steel bars.

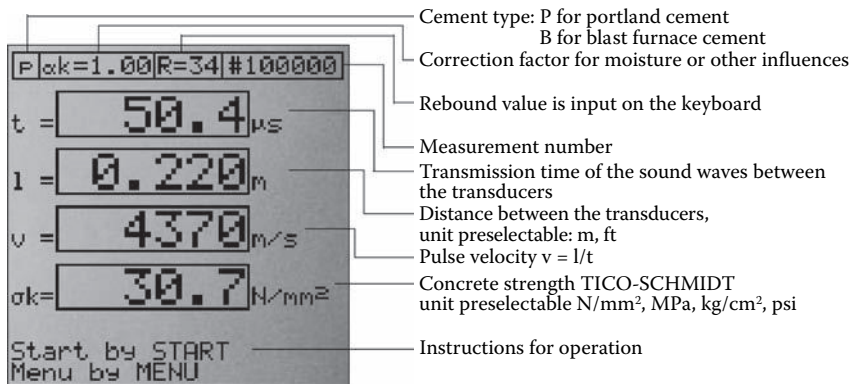


FIGURE 4.16 Ultrasonic pulse velocity screen.

TABLE 4.4 Temperature Effect on Pulse Transmission Velocity		
Percentage correction of the velocity reading (%)		Temperature (°C)
+4	+5	60
+1.7	+2	40
0	0	20
-1	-0.5	0
-7.5	-1.5	-4

TABLE 4.5 Relation between Concrete Quality and Pulse Velocity	
Pulse velocity (km/sec)	Concrete quality degree
>4.5	Excellent
4.5-3.5	Good
3.5-3.0	Fair
3.0-2.0	Poor
<2.0	Very poor

TABLE 4.6
Relation between Elastic Modulus and Pulse Velocity

Elastic modulus (MN/mm ²)		Transmission pulse velocity (km/sec)
Static	Dynamic	
13,000	24,000	3.6
15,000	26,000	3.8
18,000	29,000	4.0
22,000	32,000	4.2
27,000	36,000	4.4
34,000	42,000	4.6
43,000	49,000	4.8
52,000	58,000	5.0

4.4.2.4 Load Test for Concrete Members

This test is done in the following conditions:

- if the core test gives results of concrete compressive strength lower than characteristic concrete strength, which is defined in the design
- if this test is included in the project specifications
- if there is doubt of the ability of the concrete structure member to withstand design loads

This test is usually performed on the slabs and, in some cases, the beams. This test exposes the concrete slab to a certain load and then removes the load; during this period, it measures the deformation on the concrete member as deflection or presence of cracks and compares it with the allowable limit in the specifications.

4.4.2.4.1 I-Test Procedure

This is done by loading the concrete member with load equal to the following:

$$\text{Load} = 0.85 \text{ (1.4 dead load + 1.6 live load)} \tag{4.6}$$

Applying the load will be done by using sacks of sand or concrete blocks. In the case of sand, sacks are calibrated at at least 10 sacks for every span, about 15 m² through direct weight of the sacks. These sacks are chosen randomly to determine the weight of an average sack. The sacks are then put on the concrete member that will be tested; distance between vertical sacks needs to be considered to prevent an arching effect. As for the concrete blocks, the weight and calibration should be measured. Also, the horizontal distance left should be taken into account to avoid influence of the arching effect.

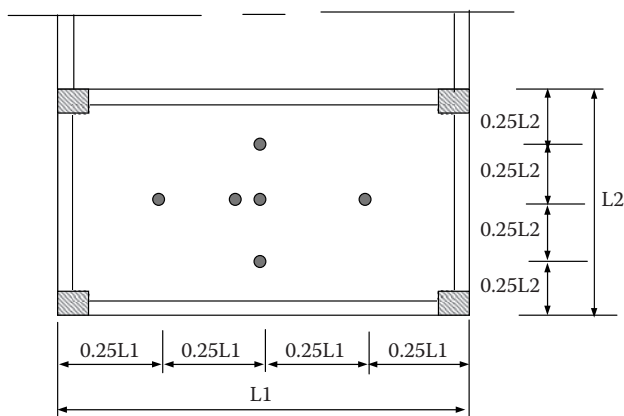


FIGURE 4.17 Location of measurement devices.

It is important to identify the adjacent elements that have an impact on the structure element to be loaded in order to obtain the maximum possible deformation for the test member. Before load processing, the location of the test must be taken into account by identifying the places where the gauges will be placed, as well as calculating the actual dead load on the concrete member, through the identification weight of the same member. In addition, coverage such as tiles, which will be installed on a slab of concrete, as well as lower coverage should be considered and plastering or weight of any kind of finishing work should be included. The location of the measurement unit is shown as an example for a slab test in figure 4.17. The figure illustrates the following specifications:

- It places it in the middle of the span; placed beside it is another as a reserve.
- Another measurement device is placed at a quarter of the span from the support; the consultant engineer must define the other reasonable location for measurement.
- The measurement devices must be calibrated and certified before use—preferably with less sensitivity of 0.01 mm; its scale is about 50 mm.
- It is necessary to have devices that measure the cracks' widths; this device must have an accuracy of 0.01 mm.

4.4.2.4.2 Test Procedure

- Define load test = 0.85 (1.4 dead load + 1.6 live load)—a dead load that has already affected the member.
- Take the reading of the deflection before starting the test (R1).
- Start with 25% of the test load and avoid the arching effect or any impact load.
- Read the measurement for the effect of 25% of the load and visually inspect the member to see if there are any cracks. If cracks are present, measure the width.
- Repeat this procedure three times; each time, increase 25% of the load.

- Record the time for the last load and the last deflection reading and crack thickness.
- After 24 hours of load affects the time record, draw the location of the cracks, the maximum thickness, and the deflection reading (R2); then remove the load gradually and avoid any impact load.
- After removing the entire load, measure the deflection reading and crack width.
- Twenty-four hours after removing the load, record the measurement and the reading (R3) and record the crack width.

4.4.2.4.3 Resulting Calculations

The maximum deflection from load effect after 24 hours is:

$$\text{Maximum deflection} = (\text{first measurement after passing 24 hours from load effect-reading before load effect}) \times \text{device sensitivity} \quad (4.7)$$

$$\text{Maximum deflection} = (R2 - R1) \times \text{device sensitivity} \quad (4.8)$$

If any problems occur in the first device use, the second device reading should be used. If the reading of the two devices is close, the average of the two readings should be taken. The remaining maximum deflection after 24 hours of completely removing the load will be from the following equation:

$$\text{Maximum remaining deflection} = (\text{reading of the device 24 hours after removing the load} - \text{reading before load effect}) \times \text{device sensitivity} \quad (4.9)$$

$$\text{Maximum remaining deflection} = (R3 - R2) \times \text{device sensitivity} \quad (4.10)$$

The maximum recovery deflection is calculated as follows:

$$\text{Maximum deflection recovery} = \text{max. deflection} - \text{max. remaining deflection} \quad (4.11)$$

The maximum deflection after 24 hours from loading and the recovery maximum deflection are as shown in figure 4.17. The relation between the load and maximum deflection in cases of loading and unloading should be drawn. The maximum crack thickness is calculated after 24 hours from load effect and 24 hours after removing the load.

4.4.2.4.4 Acceptance and Refusing Limits

Calculate the maximum allowable deflection for the member as follows:

$$\text{The maximum allowable deflection} = L^2/2t, \text{ cm} \quad (4.12)$$

where

L = the span of the member in meters—the shorter span in case of flat slab and short direction for solid slab and, in case of cantilever, twice the distance from the end of cantilever and the support face

t = thickness of the concrete member in centimeters

Compare between the maximum deflection recorded after 24 hours from load effect and the allowable maximum deflection; there will be three outcomes:

1. If, after 24 hours, the maximum deflection from load effect is less than the allowable maximum deflection from the previous equation, then the test is successful and the member can carry the load safely.
2. If, after 24 hours, the maximum deflection from load effect is higher than the allowable maximum deflection, then the recovery deflection after 24 hours from removing the load must be equal to or higher than 75% of maximum deflection (recovery deflection ≥ 0.75 maximum deflection). If this condition is verified, then the member is considered to have passed the test.
3. If the recovery is less than 75% of the maximum deflection, the test should be repeated using the same procedure, but 72 hours after removing the load from the first test.

After repeating the test for a second time using the same procedure and precautions, this concrete structure member will be refused if not verified based on the following two conditions:

1. if the recovery deflection in the second test is less than 75% of the maximum deflection after 24 hours from load effect in the second test
2. if the recorded maximum crack thickness is not allowed

4.4.2.5 Comparison between Different Tests

The different methods to determine hardened concrete strength have advantages and disadvantages, which are summarized in table 4.7. Moreover, these tests are different based on their costs, representation of the concrete member, and accuracy

TABLE 4.7
Comparison between Different Test Methods

Test method	Probable damage	Precaution requirement
Overload test	Possible member loss	Member must be isolated or allowance of distributing the load to the adjacent members Extensive safety precaution in case of collapse failure
Cores	Holes to be made good	Limitations of core size and numbers Safety precaution for critical member
Ultrasonic	None	Need two smooth surfaces
Rebound hammer	None	Need a smooth surface

TABLE 4.8
Performance Comparison for Different Methods

Test method	Damage to concrete	Representative to concrete	Accuracy	Speed of the test	Cost
Overload test	Variable	Good	Good	Slow	High
Cores	Moderate	Moderate	Good	Slow	High
Ultrasonic	None	Good	Moderate	Fast	Low
Rebound hammer	Unlikely	Surface only	Poor	Fast	Very low

TABLE 4.9
Variation between Standard Cube Strength and Actual Strength on Site

Concrete member	Actual strength/standard cube strength after 28 days	
	Average	Range
Column	0.65	0.55–0.75
Wall	0.65	0.45–0.95
Beam	0.75	0.6–1.0
Slab	0.5	0.4–0.6

of the measured strength to the actual concrete strength; this comparison is illustrated in table 4.8.

After the results of the tests are obtained, it will be found that the data on strength are lower than the concrete strength specified in the drawings or project specifications. The value of concrete strength in the drawing is the standard cube (cylinder) compressive strength after 28 days. Note that the cubes or the cylinder will be poured and compacted based on the standard; the curing process is for 28 days, so it is different from what will happen on the site. For example, the curing will not be for 28 days and the concrete slabs or columns cannot be immersed in water continuously for 28 days. The temperature on-site is surely different from and more varied than that in the laboratory. In addition, the concrete pouring and compaction method for the cubes or cylinder is different from that in the actual member size. All these variations were taken into consideration in the factors in the design codes, but the test measures the strength after complete hardening of the concrete. Table 4.9 shows a comparison between a standard cube test and *in situ* test for different reinforced concrete members.

4.4.3 SOURCES OF CONCRETE FAILURE

Corrosion is not the only source of failure. Many other sources cause deterioration on reinforced concrete structures; this must be kept in mind and understood well when an inspection is undertaken. These sources of failure include:

- unsuitable materials
 - unsound aggregate

- reactive aggregate
- contaminated aggregate
- using the wrong type of cement
- cement manufacturer error
- wrong type of admixture
- substandard admixture
- contaminated admixture
- organically contaminated water
- chemically contaminated water
- wrong kind of reinforcement
- size error of steel bars
- improper workmanship
 - faulty design
 - incorrect concrete mixture (low or high cement content and incorrect admixture dosage)
 - unstable formwork
 - misplaced reinforcement
 - error in handling and placing concrete (segregation, bad placing, and inadequate compacting)
 - curing incomplete
- environmental factors
 - soil alkali
 - seawater or sewage
 - acid industry
 - freezing and thawing
- structural factors
 - load exceeding design
 - accident as ballast load or dropped object
 - earthquake load

4.4.4 EXAMPLE FOR STRUCTURE EVALUATION

Our case study is an administration building located near the Red Sea in Egypt. The building is a steel structure constructed around 1975. It consists of two stories; the ground floor is a room for employees waiting for their flights and the first floor is an administration office of an aviation and airport control room. The structure system is steel composed of beams and columns with reinforced concrete slab on steel corrugated sheet.

The Red Sea is only about 200 m away. The first impression in the visual inspection is that there has been no maintenance for a long time as the corrugated sheet is corroded on the lower parts of the corrugated sheets outside the building for about 300 mm above the ground. The paint condition is generally good, but not as good as the original structural painting, so maintenance has been done periodically. During discussion with the owner (a very important step in any assessment), he mentioned that there has been no maintenance for around 5 years due to a shortage of funds. Near the bathroom, a sign of water can be seen on the false ceiling;



FIGURE 4.18 Corrosion in corrugated sheet.

therefore, it needs to be inspected in more detail. The false ceiling was removed and the corrugated sheet of the floor carrying the first floor was found, as shown in figure 4.18.

The challenge is to define whether the corrugated sheet, which is corroded, is carrying a load or not. If it is working with concrete as a composite section, this means that it is carrying a load and needs to be replaced. On this type of building, to do this repair by removing the slab and fixing a new composite section for the slab, it is necessary to hire a special contractor, which is expensive. Therefore, destroying the building and building a new one may be a better solution from an economic point of view. On the other hand, the corrugated sheet may be used as shuttering instead of wood and, after pouring concrete, kept as it is. Then, there would be no need to remove it. This method was famous in the 1970s; the specialized engineering and construction company is from the United States.

All the drawings and specifications do not exist, so it is necessary to collect information to confirm the structural system and whether either of these two methods of construction was used. If the slab is a composite section, the steel reinforcement will be light and always be a steel wire mesh for shrinkage. On the other hand, if it is only shuttering, it needs enough steel bars to carry the load as it will be designed as reinforced concrete slab.

Information was collected from the engineers who reviewed the design previously and others who supervised the building and a hole was made to see the steel bars; all these data confirm that there are steel bars of 13 mm and more, so this corrugated sheet is not needed to carry the load. Because the corrugated sheet is used as shuttering only, the building can exist longer.

In brief, an engineer should take his time in evaluation and not jump to a conclusion from his first look. He should not hesitate to collect data from any source; if they do not add value, they will improve the data he already has. The decision is very critical; in this case, there are two different options: to destroy the building or to keep it for possibly another 30 years. Imagine the consequences if a wrong decision is made!

4.4.5 EXAMPLE FOR STRUCTURE ASSESSMENT

There are cracks on the ring beam around the steel tanks. This tank is carrying oil mixed with a high percentage of water. The tanks are not located in a very bad environmental situation. They were constructed in about 1983; a photo is shown in figure 4.19a. When an observation is made, a leak is found on the tanks that contain hydrocarbon and waters that affect the reinforced concrete beam. As shown in figure 4.19b, the corrosion is parallel to the stirrups, every 200 mm and on the corner steel bars.



FIGURE 4.19a General tank view.



FIGURE 4.19b Cracks on the tank ring beam.

Therefore, reason and the shape of the cracks revealed that there is corrosion on the steel bars. In figures 4.19c and 4.19d, some areas have a spalling of concrete cover and the reduction on the steel bar cross-sectional area is more than 20%; therefore, it needs complete repair. From a structural point of view, the steel bars have a tension force as the ring beam is always under tension. The repair procedure for this tank is described in detail in chapter 6.



FIGURE 4.19c Spalling of the concrete cover.



FIGURE 4.19d Reduction in steel cross-sectional area.

4.5 TEST METHODS FOR CORRODED STEEL IN CONCRETE

During the corrosion process, the volume of steel bars is increased and, due to volume increase, internal stresses form and cause the concrete to crack. These cracks may be vertical or horizontal and they increase in length and width until the concrete cover falls. It is necessary to know exactly the parts of concrete elements that cause a separation between concrete cover and concrete itself.

4.5.1 MANUAL METHOD

The deteriorated area can be defined by using a hammer to hit the concrete cover. If the sound that is made indicates air behind the surface of the concrete cover, the steel bars are corroded and concrete cover separation is a possibility. For large areas of concrete surface, such as concrete bridge decks, a steel chain can be moved on the surface to define the separated cover and the damaged parts. There are more complicated methods than using hammer or steel chain, such as using infrared or ultrasound.

Using the preceding inspection methods, an engineer can define the area that is expected to be corroded and predict concrete cover spall after a short time; this must be considered in the next repair. A manual hammer or steel chain is used in this way on the surface deck of the bridge. This method is fast, easy to use, and less expensive than other methods such as radar, ultrasound, or infrared rays; this method can be used in big structures or when evaluating huge areas such as bridge decks.

The use of the hammer is often associated with the process of visual inspection. The hammer is easily carried, low cost, and quick to locate defects. It can make marks at the places where the sound indicates a vacuum behind the concrete surface. Use of infrared ray equipment is affected by the temperature; the best accuracy occurs when the temperature is as warm or as cold as possible. Through the camera in the infrared the engineer can clarify where the layer of concrete that separated from the concrete has a temperature different from that of the main concrete part, as shown in figure 4.20. Infrared is used on the surfaces of bridges in the early morning or in the evening, when the air is clean and pure, because then the area of separation between concrete cover and concrete can be identified.

Radar is also used in determining areas that have been separated from the concrete cover of a concrete structure; the United States uses a system of radar mounted on wheels to inspect bridge surfaces. Figure 4.21 illustrates radar fitted on a vehicle inspecting the bridge surface. In Europe and the United Kingdom, radar is activated manually and used in buildings and different structures rather than on bridges. Many sources have studied this equipment (Blingy 1993; Cady and Gannon 1992; Titman 1993). Note that radar is not accurate in defining the size and location of an area that has a separation on the concrete cover. Rather, it generally defines all the separated parts on the bridge as a whole (Alongi, Clemena, and Cady 1993). In the United States radar is used together with infrared to increase the accuracy in defining a defective area, but this is expensive. Note also that infrared is used less frequently because that method needs reasonable temperatures during use.

Generally, it is also worth noting that these methods, whether manual or using radar and infrared, need workers trained professionally, with high levels of experience and competence in reading the measurement and defining the defective area

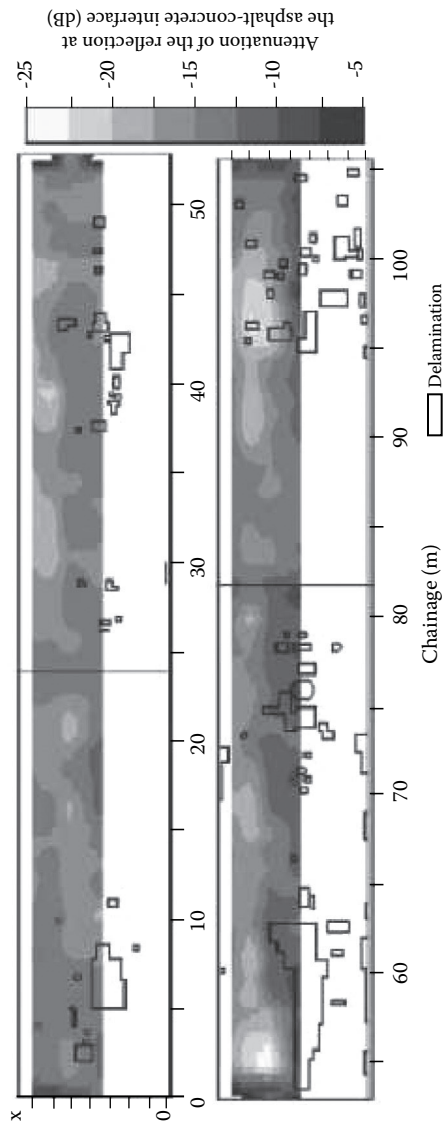


FIGURE 4.20 Measurement results to a bridge deck.

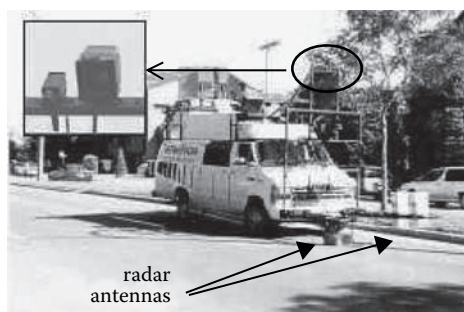


FIGURE 4.21 Radar antennas.

accurately. There are precautions and limits on the use of these methods: When water is found in the cracks, when the separation is deep inside the concrete, and when the steel bars are embedded with depth within the concrete, it will be difficult to define the separate areas as in structures like bridges that are always noisy due to moving cars. Therefore, a hammer cannot be used easily to define the defective area. Moreover, the existence of water and the depth of separation affect the readings in the case of radar and infrared.

It is worth mentioning that, in the execution of the repair process, an area of a larger size than that of the defective area defined before by the different methods should be removed, because there is time between structure evaluation and start of repair due to administration work, finding the funds, and obtaining approval. This duration is sufficient to increase the volume of concrete, which causes more separation.

In brief, the use of radar is good in cases of estimating the volume on the surface of a bridge collapse as a whole, but not appropriate to locate the separation of concrete accurately. The main disadvantage of use of infrared is that it needs appropriate weather conditions to provide reasonable accuracy.

4.5.2 CONCRETE COVER MEASUREMENTS

The thickness of the concrete cover measurement is used in modern construction so as to ensure that the thickness is in conformity with specifications (explained in chapter 5) in order to protect the steel from corrosion. The process of measuring the thickness of the concrete cover in structures was marked by the beginning of the corrosion as the lack of concrete cover thickness increases the corrosion rate for corrosion as a result of chlorides or carbonation. This expedites the propagation inside the concrete, causing the speed of steel corrosion; also, the lack of cover helps in spreading moisture and oxygen, which form the main basis for the corrosion process.

The measurement of the concrete cover thickness explains the causes of corrosion and identifies areas that have the capability to corrode faster. This measurement (Allred 1993) needs to be defined for axes Y and X in order to determine the thickness of the concrete cover at every point on the structure. The equipment that measures this thickness is simple and high tech; measurement readings can be obtained as numbers. Figure 4.22 illustrates use of electromagnetic cover meter equipment on a bridge deck. Figure 4.23 presents equipment shape and the method of reading.

Radiographs can be used for bridges, but the cost is high (Bungey 1993; Cady and Gannon 1992). The magnetic cover method is a simple method, but it is affected by the distance between steel bars. The thickness of the concrete cover exerts a large influence on the readings. Since this method depends on the supply of electricity



FIGURE 4.22 Concrete cover measurement to a bridge deck.



FIGURE 4.23 Concrete cover thickness measurement machine.

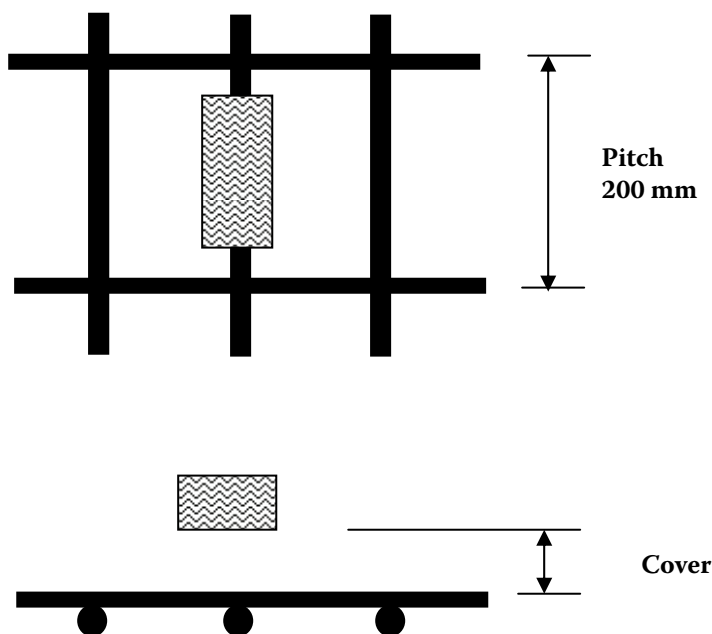


FIGURE 4.24 Device location on a concrete surface.

through a 9-V battery, while the second side of measurement for the potential voltage envelope when completed by the steel bars buried, as shown in figure 4.24.

The British standard is the only standard concerned with measurement of concrete cover after construction (in BS 1881, Part 204). In 1993, Alldred studied the accuracy of the measurement of the cover when there are more steel bars close to each other. He suggested using more than one head of measurement as that would increase the accuracy of the reading and the small heads have an impact on the accuracy of the equipment. Therefore, the problem with this method is that dense steel reinforcement in the concrete section will give inaccurate data; in this case, the equipment will be calibrated based on the existing steel bars as the reading is affected by the type of steel. The person who works on the equipment must be competent and aware of anything that can affect the reading, such as bolts, steel wires, etc. (see figures 4.25 and 4.26).

4.5.3 HALF-CELL POTENTIAL MEASUREMENTS

This system is used to determine bars that have lost passive protection. As explained in chapter 2, the bars in the concrete that have lost passive protection were exposed to carbonating or existing chlorides, causing chemicals to react with the alkaline around the steel, which loses its passive layer. The half-cell equipment (ASTM C 876 1991) is composed of a rod of metal in solution by the same metal ions as a copper rod in saturated solution of copper sulfate. This is considered one of the most common types of equipment; some other equipment, such as bars of silver in silver chloride solution, can be linked through the voltmeter and other party potentiometers connected to the steel bars, as in figure 4.27.



FIGURE 4.25 The device right location.

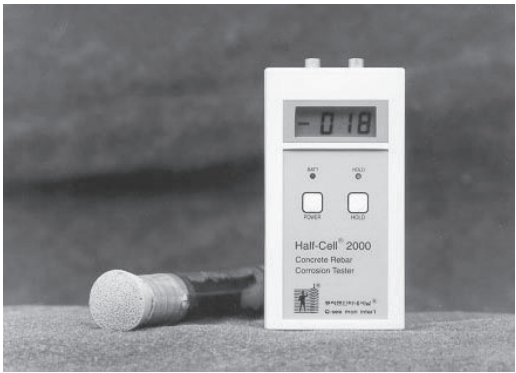


FIGURE 4.26 Half cell equipment.

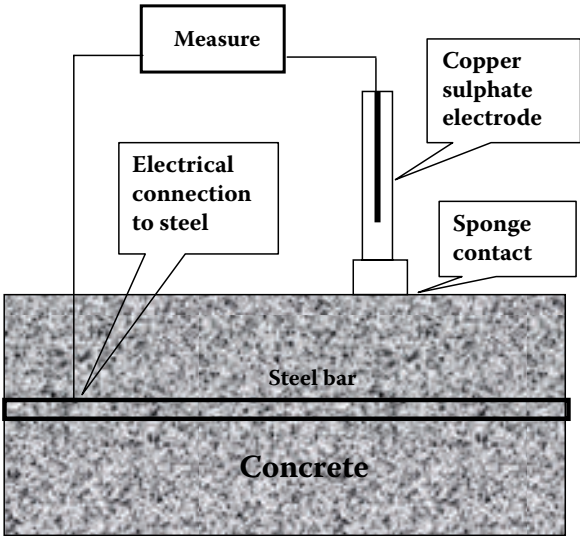


FIGURE 4.27 Drawing for half-cell device measurement.

TABLE 4.10**ASTM Specification for Steel Corrosion for Different Half-Cells**

Silver/silver chloride	Copper/copper sulphate	Standard hydrogen electrode	Calomel	Corrosion condition
Greater than –106 mV	Greater than –200 mV	Greater than +116 mV	Greater than –126 mV	Low (10% risk of corrosion)
–106 to –256 mV	–200 to –350 mV	+116 to –34 mV	–126 to –276 mV	Intermediate corrosion risk
Less than –256 mV	Less than –350 mV	Less than –34 mV	Less than –276 mV	High (<90% risk of corrosion)
Less than –406 mV	Less than –500 mV	Less than –184 mV	Less than –426 mV	Severe corrosion

If the steel bars have a passive protection layer, the potential volts are small—0 to 200 mV copper/copper sulfate. If there has been a breakdown of the passive protection layer and some quantity of steel melted as a result of movement of ions, the potential voltage is about 350 mV. When the value is higher than this, the steel has already started to corrode. It is recommended to use half-cell equipment consisting of silver and silver chloride solution or mercury and mercury oxide solution. The equipment that consists of copper and copper sulfate solution is used, but is not recommended because it requires regular maintenance and thus some readings will be erroneous.

To survey a large surface area of concrete to determine the bars that have been corroded, several half-cells on different locations on the surface need to be installed. These will be designated by the inspector. The gathering data will be represented as a contour line on the surface by which the location of the corroded bars underneath this large surface can be defined.

The measurement of the half-cell to the steel reinforcement bars is to define the probability of causing corrosion to the steel bars. The ASTM C867 specification presents a method to clarify the data obtained from measuring the potential voltage. These values are shown in table 4.10 for copper/copper sulfate and silver/chloride silver equipment. The condition of corrosion is explained in each case of corresponding value.

The inaccuracy of such measurements is a result of the presence of water, which increases the negative values without corrosion occurring in steel, as wetting columns and walls gives high negative values for potential voltage regardless of the degree of steel corrosion. Negative voltage differs significantly underwater in off-shore structures; in spite of the lack of oxygen, it reduces the rate of corrosion.

4.5.4 ELECTRICAL RESISTIVITY MEASUREMENT

The process of steel corrosion is the process of electrical and chemical reactions as electrical resistance of concrete is a major factor in the rate of steel corrosion. The corrosion rate in the steel reinforcement depends on the movement of ions from the

anode to the cathode. A four-probe resistivity operation is used to measure the resistance of the soil to electricity; then, some changes are made to this system to measure the electrical resistance of concrete. This equipment is shown in figure 4.28.

The measured resistance of concrete by the four-probe operation passes electric current between two outside probes and measures the potential voltage between the inner two probes, as illustrated in figure 4.29. From this, an engineer can measure



FIGURE 4.28 Electrical resistivity measurement equipment.

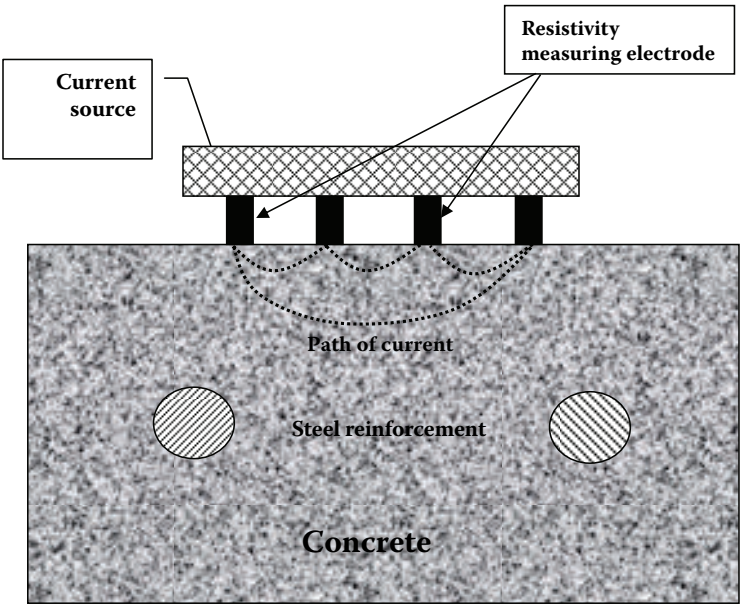


FIGURE 4.29 The work flow of electrical resistivity measurement equipment.

the electrical resistivity of the concrete. Some equipment is needed to make a hole in the concrete to fix the probes, but the new equipment puts it on the surface only.

The electrical resistance is greatly affected by the moisture in the internal porous concrete as well as by the quality of the content—that is, the w/c ratio in the concrete mix, curing, and using additives. Note that the level of chloride does not have much impact on the electrical resistance of concrete.

During the measurement, there must be a greater distance between the probes of the largest size of aggregate; measuring the electrical resistance of only a piece of aggregate is avoided. It is possible to measure away from the steel bar, so the readings must be taken perpendicular to the steel reinforcement. Figure 4.29 illustrates an appropriate position for taking readings.

The measurement of electrical resistance of the concrete cover occurs only through the use of two probes. The first probe is in the steel reinforcement and the second moves in the concrete surface. This is shown by the results obtained on the electrical resistance of concrete and can be distinguished through the comparison that follows, according to Broomfield, Langford, and McAnoy (1987) and Broomfield et al. (1993):

- >120 k Ω -cm: low corrosion rate
- 10–120 k Ω -cm: low to medium corrosion rate
- 5–10 k Ω -cm: high corrosion rate
- <5 k Ω -cm: very high corrosion rate

Many researchers have studied the relation between electrical resistivity and corrosion rate by using electron probes and have concluded the following:

- >100 k Ω -cm: cannot differentiate between passive and active steel reinforcement
- 50–100 k Ω -cm: slow corrosion rate
- 10–50 k Ω -cm: from medium to high corrosion rate with active steel to corrosion
- <10 k Ω -cm: no impact of electrical resistivity on corrosion process

From the previous relation, this method is considered a reasonable way to know the corrosion rate in steel reinforcement.

The electrical resistance is expressed on the corrosion rate of steel reinforcement and also is a good representative of the concrete quality control and density, which depends on the compaction method on site. But, according to a study from Liverpool University, using four poles to measure the electrical resistance is incorrect because of the effect of steel on the reading. If the angle of measurement is perpendicular to the steel, the reading error can be reduced to the least extent possible. The study also found electrical resistance at more than 10 k Ω -cm. Active or passive protection in reinforced steel cannot be differentiated; if electrical resistance is 10 km Ω -cm, the resistance is not effective in the process of corrosion.

4.5.5 MEASUREMENT OF CARBONATION DEPTH

Carbonation depth is formed in the propagation of carbon dioxide inside the cover of the concrete, changing the concrete alkalinity value and reducing its value. The concrete alkalinity value is expressed by pH values equal to 12–13, with carbon dioxide propagation causing a reaction that reduces the alkalinity. This will affect the steel surface and then damage the passive protection layer around the steel bars; the corrosion process will then start.

Therefore, it is very important to define the depth of the concrete that transforms to carbonation and how far away from or close to the steel bars it is. This test (Parrott 1987) is performed by spraying the surface of broken concrete or breaking it using special tools to obtain the carbonation depth by phenolphthalein dissolved in alcohol. This solution color becomes pink when it touches the surface of concrete with alkalinity value of pH of about 12–13.5, and the color turns gray or blue if concrete loses its alkalinity and pH value is less than 9. In this case, steel bars are losing the passive protection layer. The last case loses alkalinity and the pH is less than 9.

It is necessary that the part to be tested be newly broken whether it is a beam, column, or slab. After measuring the carbonation depth in the concrete cover and the distance from carbonation depth to the steel bar, whether it reaches the steel bars or not, it is easy to evaluate the corrosion risk for the steel reinforcement.

The best solution is to use phenolphthalein with alcohol and water: 1 mg phenolphthalein with 100 mm alcohol/water (50:50 mixing ratio) or more alcohol than water (Building Research Center 1981; Parrott 1987) if the concrete is completely dry, thus humidifying the surface by water. If the thickness of carbonation is about 5–10 mm, it is going to break the passive layer to the steel bar 5 mm from the presence of changed color of the concrete surface.

Some of the aggregate or concrete mixtures have a dark color, making reading phenolphthalein very difficult; therefore, the surface should be clean when testing takes place. This test must be done in the areas accessible to the work of the breakers required in the concrete, as well as facilitate the work of repair for the broken part.

4.5.6 CHLORIDE TESTS

Chloride tests (AASHTO T260-84 1984; ASTM D 1411-82 1982) rely on an analysis of the samples of the concrete powder to determine the quantity of chloride. This test is done by using a drilling machine to rotary dig inside the concrete and extract concrete powder from the hole or through the broken part of the concrete. Several separate samples must be taken at different depths. The depth of the hole will vary to increase the accuracy and samples are always taken from 2–5 mm, first 5 mm from the surface. Often the chloride concentration is very high, particularly in structures exposed to seawater and chlorides.

The holes are made by special devices that collect the concrete powder product that results from drilling the hole. The concrete powder will be taken at every drilling depth and will be added to a solution of acid, which is determined by the amount of chloride concentration. There are two principal ways of measuring *in situ* (quanta strips); the method to determine the electron–ion (specific ion electron) and

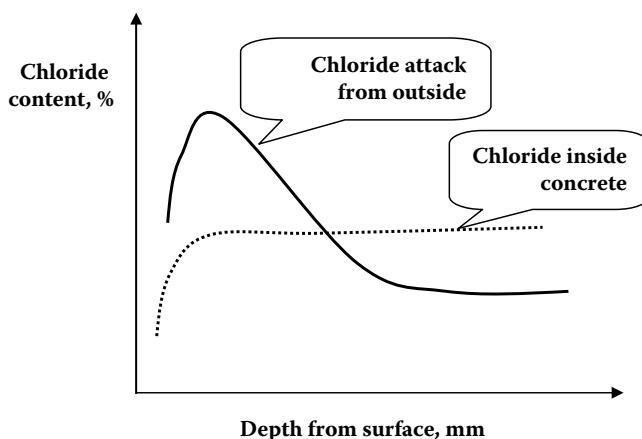


FIGURE 4.30 Comparison between a chloride attack from outside the concrete and inside the concrete.

the recent experience is more accurate, but needs expensive equipment. Experiments of chloride concentration need trained professionals to conduct them.

When the readings of chloride concentration at various depths have been assembled, draw the shape to chloride concentration with depth from the surface of concrete into the concrete. From the general figure (figure 4.30), it can be determined whether chlorides are within the concrete or from the impact of air and environmental factors affecting the concrete. Figure 4.30 shows the difference between the two cases.

From the previous figure, it can be seen that, in the case of the impact of chlorides from outside, in the beginning the chloride concentration is high and then gradually decreases with depth until it reaches constant value. On the other hand, if the chloride exists inside the concrete, such as during concrete mixing due to use of salt water or an aggregate with high chloride content, the figure shows that near the surface the chloride concentration is minimal and increases with depth until it reaches constant values with the depth.

The chloride test method is done through melted acid and calibration work after that, based on specifications (BS 1881, Part 124). In addition to the method of melting, the melting acid can work in water; these tests follow ASTM D 1411-82 (1982) and AASHTO T260 (1984). The effect of chloride concentration on the steel reinforcement will be defined if the chloride concentration exceeds 0.6 from hydroxyl concentration, which causes failure in the passive layer around the steel reinforcement. This ratio is approximately 0.2–0.4% chloride by weight of cement, 1 lb/yd³ concrete, or 0.5% chloride by the weight of concrete. The figure for the distribution of chloride concentration with depth can determine the degree of impact of chloride in the steel reinforcement and the impact of chloride whether inside or outside the concrete.

4.6 BUILDING ASSESSMENT

The tests that have been detailed in the preceding sections are only tools to assist in the process of assessing a building. The main factor in the process—important to

establishing the building assessment—is the experience of the engineer examining the structure. The evaluation of the building determines whether the structure is or is not subject to repair (e.g., because the situation is entirely bad or the cost of repair is near the cost of demolition and reconstruction).

The evaluation of the building is vital and critical to the decision process as decision making will be based on an assessment of the building in terms of safety and of the repair cost. Therefore, the scope of repair work necessary for the building is based on the accuracy of the test results and the experience of the skilled labor force who conducts these tests. When the method of repair and its necessary cost are being determined, it is important to consider the methods required to protect the structure from corrosion after repair. Different ways of protection and their advantages and disadvantages will be illustrated in chapter 7.

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5 Codes and Specification Guides

5.1 INTRODUCTION

Now we are going to discuss an issue of great importance as it directly affects the life of concrete structures: the design and construction specifications that should be followed in different codes to avoid corrosion of the steel reinforcement in concrete. If we want to construct a new building, we put a lot of effort into solving problems of and paying money for construction. Thus, we must take extreme care to preserve this investment or we will find ourselves having to put more effort and money into maintaining it.

The proverb “an ounce of prevention is worth a pound of cure” fits the concept that codes and specifications provide us the means to control the corrosion of steel reinforcement in concrete. When a new building is constructed, it is important to follow all the precautions in design and construction on-site to prevent corrosion of steel in concrete. This is better than constructing a building that does not follow standards and then faces many problems in the future. This could mean going through costly repairs and, in some cases, not returning the building to its original strength.

Most different codes and technical specifications that control concrete design and construction work to minimize the risk of corrosion in the steel reinforcements in the following ways:

- Define maximum allowable limit of chloride in concrete.
- Define components and mixture of concrete cover.
- Define concrete cover thickness.
- Define maximum allowable crack width based on design method.

Most specifications take into consideration the method of construction—for example, if the concrete is cast *in situ*, precast, or prestressed—as well as weather conditions that might affect the structure. Structures that have been exposed to severe corrosion include marine structures that have been directly exposed to seawater, such as ports, offshore platforms used in the oil industry, and bridge supports. These structures are highly exposed to corrosion, especially the parts exposed periodically to dry cycles and cycles of seawater. These parts make up the “splash zone” and are considered one of the most dangerous elements of the structures, where the possibility of steel corrosion is very high.

One other structure frequently prone to corrosion is concrete bridges located in cold regions, such as those in Europe and some parts of North America. These countries use salt to melt the ice.

In the Middle East, in addition to buildings exposed to seawater, the climate has a very high relative humidity and high temperatures, which may cause corrosion of steel very quickly, as we explained in chapter 3.

Therefore, the different specifications of each country have always been concentrated on that country’s environmental conditions, as well as methods of quality control for concrete used in construction and the construction method commonly used in the country. Hence, it is clear that each country has special precautions in design, provided to the designer by the code, and specifications consistent with the weather and its environmental factors so as to avoid corrosion in steel reinforcement as much as possible.

This chapter will illustrate the specifications used in American, Egyptian, and British code. These codes have special conditions concerning the percentage of chlorides in concrete and thickness of the concrete cover, as well as other specifications. These should be followed precisely to avoid the risk of steel corrosion.

5.2 ALLOWABLE CHLORIDE CONTENT IN CONCRETE

The majority of international specifications set maximum amounts of chloride content in reinforced concrete because of its adverse impact on steel, as has been explained before. The sources of the chlorides in the concrete industry are often additives, some coarse aggregate or sand that may contain salts, or the water used in the concrete mix. The percentage of chloride available in cement is very small and is controlled by the cement industry itself.

The specifications of the American Concrete Industry (ACI) place limits on the level of chlorides in reinforced concrete, as shown in table 5.1. However, the American code shows different limits for the content of chlorides in concrete. It puts limits on the content of dissolved ions of chlorides (ACI 318 R-89 1994) or chloride ions

TABLE 5.1
Recommendation of Maximum Chloride Ion Content to Protect from
Corrosion According to ACI Code

Structure member type	Maximum chloride content (% of cement weight)			
	Soluble in water ^a	Total ^b	Soluble in acid ^c	Total ^d
Prestressed concrete	0.06		0.06	0.08
Reinforced concrete exposed to chloride in service	0.15	0.1	0.1	0.2
Reinforced concrete in dry conditions or protected from moisture in service	1.0			
Another fixed member	0.3	0.15		

^a ACI 318R-89 9) ACI building code.
^b ACI Committee 221.
^c ACI Committee 357.
^d ACI Committee 222.

TABLE 5.2
Maximum Soluble Ion Content to Protect from Steel Bar Corrosion

Maximum soluble chloride ion content in water in concrete (% of cement weight)	Surrounding conditions
0.15	Reinforced concrete exposed to chlorides
1.0	Reinforced concrete dry and protected from humidity during usage
0.3	Other structure members

within the total (ACI Committee 201, 222, 357 1994) as a result of the components of concrete or due to chloride content resulting from chloride penetrating throughout the structure’s lifetime. In American specifications, ACI Committee 357 recommended that the water used in the concrete mix not contain chloride amounts of more than 0.07% in the case of reinforced concrete, or 0.04% for prestressed concrete.

In the European Union code (ENV 206 1992), the limits of chloride in concrete are identified by the type of application; the limits are 0.01% by the weight of cement in the case of plain concrete, 0.04% of the weight of cement in the case of reinforced concrete, and 0.02% of the weight of cement for prestressed concrete. These specifications prevent the use of any additives containing chlorides or using calcium chlorides in reinforced or prestressed concrete.

In the Egyptian codes, the chloride ions in hardening concrete (result of water, aggregate, cement, and additives) at the age of 28 days are not to exceed the limits shown in table 5.2. Egyptian code also recommends not using calcium chloride additives or any additives with a base from chlorides for reinforced concrete, prestressed concrete, or concrete embedded with metal. Chloride ion content in the additives must not exceed 2% by weight of additives or 0.03% by weight of cement for reinforced concrete, prestressed concrete, or portland cement with sulfate-resistant cement or cement with high sulfate resistance.

These limits of chloride content in concrete can be achieved by testing the additives before use to ensure that the chloride is within limits and does not use water that has salt in mixing or curing. Moreover, the percentage of chloride in aggregate must be within the limits; the aggregate is usually washed by water before using it in the mix to clean it of salt and remove any other fine materials.

5.3 CONCRETE COVER SPECIFICATIONS

The concrete cover is the first line of defense to protect the steel reinforcement from corrosion; therefore, complete attention should be paid to the thickness of the concrete cover. This thickness must not exceed a certain limit, according to the types of structure members and surrounding environmental conditions. Therefore, many codes have devoted specifications to the thickness of concrete cover according to the nature of the structure, method of construction, and quality of concrete used, as well as weather factors surrounding the structure.

5.3.1 BRITISH STANDARDS

In British standards, the ability of the concrete cover to protect the reinforcement steel bars from corrosion depends on the thickness of the concrete cover and the quality of the concrete. Moreover, when we talk about quality control of concrete—particularly, concrete cover—we mean that the concrete has no cracks and good compaction since it will have a high density and a low water-to-cement (w/c) ratio. This works to prevent the ability of water to permeate easily into the concrete or any other material that will have an impact on steel reinforcement and cause corrosion.

Therefore, it is logical that concrete cover thickness is a function of the expected concrete quality; for example, in the case of a high-quality concrete, there is little need for a thick cover. This philosophy depends on that in British standard BS 8110 (1985). The specifications of the concrete cover thickness in this British code are indicated in table 5.3. This depends on the weather factors to which the concrete structure is exposed, as well as the concrete strength and its quality based on the cement content and w/c ratio.

5.3.2 AMERICAN CODES

A specification of the American ACI code does not give the exact details, as in the British code, but the least thickness of the concrete cover for concrete that has been poured at the site is shown in table 5.4. To produce good concrete from that exposed to water with a high salinity ratio, such as seawater, the American code sets the maximum w/c ratio equal to 0.4. In the case of structures exposed to seawater, the least allowable thickness of the concrete cover is 50 mm. Because of mistakes expected

TABLE 5.3
Properties and Thickness of Concrete Cover in BS 8110

	Concrete cover thickness				
	30	35	40	45	50
Concrete grade (MPa)					
Water-to-cement ratio	0.65	0.65	0.55	0.50	0.45
Minimum cement content (kg/m ³)	275	300	325	350	400
Environmental conditions					
Moderate: concrete surface protected from external weather or hard condition	25	20	20	20	20
Moderate: concrete surface protected from rain or freezing and the concrete under water or the concrete adjacent to no affected soil		35	30	25	20
Hard: concrete surface exposed to rain and wetting and dry			40	30	25
Very hard: concrete exposed to seawater spray or melting ice by salt or freezing			50	40	30
Maximum condition: concrete surface exposed to abrasion as seawater has solid particles, or moving water with pH 4.5 or machines or cars				60	50

TABLE 5.4
Minimum Cover Thickness for Cast-in-Place Concrete^a

Minimum cover (mm)	Type of structure
75	Concrete deposited against the ground
Formed surfaces exposed to weather or in contact with ground	
50	No. 6 bar or greater
38	No. 5 bar or smaller
Formed surfaces not exposed to weather or in contact with ground	
38	Beams, girders, and columns
19	Slabs and walls, no. 11 bar or smaller
38	Slabs and walls, nos. 14 and 18 bars

^a Per the ACI Committee 301.

during construction, it is preferred that the concrete cover design call for thickness of 65 mm so that, after execution, the least thickness of the concrete cover will be 50 mm. In the case of precast concrete, American code allows calculating concrete cover thickness that is lower than that shown in table 5.4.

5.3.3 EUROPEAN CODE

The European Union code gives precise and detailed recommendations and defines the degree of concrete strength required based on the weather conditions to which the structure is exposed. European code ENV 206 (1992) sets a ratio of water to cement, as well as the lowest permissible content of the cement in concrete and least thickness of the concrete cover corresponding to the concrete strength according to weather conditions (table 5.5). All choices will be according to the weather factors. This code has set specifications for the reinforced concrete structure and prestressed concrete.

Note that the last column in table 5.5, which determines the concrete strength characteristics, is set in megapascals (MPa); the first number is the value of the strength characteristics of cubes and the second number is the equivalent of the cylinders. For example, C30/37 means concrete characteristic cube compressive strength of 30 MPa and concrete characteristic cylinder compressive strength of 37 MPa. It is preferable to use cement resistance to sulfate sulfur if the sulfate content is more than 500 mg/kg in water or more than 3000 mg/kg in the soil. In both cases, additional painting on the concrete surface is recommended.

We must bear in mind that, during construction, the concrete cover thickness is often less than what the design calls for. Browne, Geoghegan, and Baker (1993) explored this topic and found the average thickness of the concrete cover to be about 13.9 mm, which is about half the value of that which was stated in the design (25 mm). Van Daveer (1975) has done a survey on the thickness of the concrete cover in the design of bridges. If the design stated that the concrete cover thickness should be about 38 mm, it was found that the standard deviation of the thickness of the concrete cover was very high—up to 9.5 mm. As a result, in 1997 Arnon Bentur (Bentur and

TABLE 5.5
Required Concrete Durability^a

Exposure condition	Maximum w/c ratio	Minimum cement content (kg/m ³)	Minimum concrete cover (mm)	Concrete grade
Dry	0.65	260	15	C30/37
Humid:				
No frost	0.60	280	20	C30/37
Frost	0.55	280	25	C35/45
De-icing salts	0.50	300	40	C35/45
Seawater:				
No frost	0.55	300	40	C35/45
Frost	0.50	300	40	C35/45
Aggressive chemical:				
Slightly	0.55	280	25	
Moderately	0.50	300	30	
Highly	0.45	300	40	

^a Per European code ENV 206 (1992) and British specifications recommendation DDENV 206.

TABLE 5.6
ACI Committee 357 Recommendation for Concrete Strength and Cover Thickness in Offshore Structures

Cover thickness		Minimum concrete strength at 28 days	Maximum w/c ratio	Location
Prestressed	Reinforced steel			
75	50	35	0.40	Air
90	65	35	0.40	Splash zone
75	50	35	0.45	Immersed in water

TABLE 5.7
Comparison of Different Specifications to Concrete Design in Splash Zone

Code	Concrete cover thickness (mm)	Maximum crack width (mm)	Maximum w/c ratio	Minimum cement content (kg/m ³)	Permeability factor (m/s)
DNV	50		0.45	400	10–12
FIP	75	0.004 × thickness or 0.3	0.45	400	
BS6235	75	0.004 × thickness or 0.3	0.40	400	
ACI	65		0.40	360	

Jaegermann 1989, 1990) suggested that, to obtain a 50-mm thickness of concrete cover at a site, it was necessary to call for cover thickness of about 70 mm in the design, the construction drawings, and the specifications.

The European code identified the previous deviation during construction, as the minimum concrete cover must increase by the allowable deviation. Its value is from 0–5 mm in cases of precast concrete and from 5–10 mm in cases of concrete cast *in situ*.

5.3.4 SPECIFICATIONS FOR STRUCTURES EXPOSED TO VERY SEVERE CONDITIONS

These are offshore structures that are directly exposed to seawater, such as ports or offshore platforms in the oil industry located on the sea or ocean. Based on their conditions, ACI Committee 357 set certain specifications for them. The concrete cover thickness is defined based on the construction method—that is, whether it is made of reinforced concrete or prestressed concrete. In addition to that, limits of concrete strength and w/c ratio are stated in this specification, as shown in table 5.6. The most dangerous area for corrosion in such structures is the splash zone, which is found in the region of the structure periodically exposed to seawater; here, it is not completely submerged in water, but not exposed only to air. There are several specifications for reinforced concrete in that particular region, as shown in table 5.7.

British specifications have been identified for more detailed concrete cover thickness and concrete specifications required for private structures. Moreover, a correspondence between the degree of mixing and the quality of concrete and the chloride diffusion factor in the concrete was identified; this was also in accordance with the structure's life expectancy, as indicated in table 5.8.

It is worth mentioning that the ability of the concrete cover to protect steel from corrosion depends not only on the thickness of the cover, but also on the w/c ratio, the content of cement in the mix, and the degree of quality control of concrete. While these factors are the most important influences, protection from corrosion also depends on the method of mixing, coarse aggregate and sand sieve analysis, the way in which the concrete was compacted, and the curing of concrete after pouring.

5.3.5 EGYPTIAN CODE

The Egyptian code identified the thickness of the concrete cover, which is required under environmental conditions surrounding the structure. In this code, the concrete cover thickness depends on the surface of concrete that has tension stresses; the effects of environmental factors have been divided into four sections, as shown in table 5.9. From this table, one can accurately determine the structure under any type of element. By knowing strength characteristics of the concrete construction and the type of element, one can determine the thickness of the concrete cover, as seen in table 5.10. Egyptian code states that, in any case, thickness of the concrete cover should not be less than the largest bar diameter used.

TABLE 5.8

British Code Requirement, Expected Diffusion Values, and Expected Time of Beginning of Corrosion,^a as well as Corrosion Spread^b

Source	Exposed degree	Chloride exposed condition	Concrete mix detail			Minimum concrete cover (mm)	Age (years)		
			Cement content (kg/m ³)	Maximum w/c ratio	Minimum slump (mm)		Diffusion coefficient, (m ² /s)	Ci = 0.4%	Cp = 1%
All structures BS 8110	Very severe	Spray seawater or de-icing	325	0.55	40	50	3.93 × 10 ⁻¹²	3.1	5.6
			350	0.50	45	40	3.18 × 10 ⁻¹²	2.6	4.6
			400	0.45	50	30	2.57 × 10 ⁻¹²	1.9	3.7
Bridges BS 5400 Part 4	Severe	Abrasion and sea-water contain solids	350	0.50	45	60	3.18 × 10 ⁻¹²	5.8	10.4
	Very severe	De-icing or seawater spray	400	0.45	50	50	2.57 × 10 ⁻¹²	5.4	10.2
			360	N/A	40	50	3.93 × 10 ⁻¹²	3.3	6.1
Offshore structure BS 6349 Part 1	Always submerged	seawater spray	330	0.45	50	40	2.57 × 10 ⁻¹²	3.0	5.5
		Abrasion by seawater	360	N/A	40	50	3.93 × 10 ⁻¹²	5.5	10.3
		Below sea level by 1 m	330	0.45	50	40	2.57 × 10 ⁻¹²	5.7	10.3
Seawater ENV06	Tidal/splash	Less than lowest level by 1 m	350	0.50	n/a	>50, prefer 75	3.18 × 10 ⁻¹²	3.3	7.2
		Air saturated by water	400	0.45	N/A	>50, prefer 75	3.18 × 10 ⁻¹²	7.4	16.2
		Submerged in water	400	0.45	N/A	>50, prefer 75	2.57 × 10 ⁻¹²	5.4	10.2
Chlorides rather than seawater	XS1	Spray or tidal	330	0.50	40	35	2.57 × 10 ⁻¹²	12.0	22.9
	XS2	Wet and rarely dry	330	0.50	40	35	3.93 × 10 ⁻¹²	1.5	2.7
	XS3	Wet and dry cycles	350	0.45	45	40	3.93 × 10 ⁻¹²	2.0	3.6
Chlorides rather than seawater	XS4	Wet and dry cycles	300	0.55	40	40	3.18 × 10 ⁻¹²	2.1	4.6
	XS5	Wet and dry cycles	330	0.50	40	40	3.93 × 10 ⁻¹²	2.0	3.4

^a C_i = 0.4 by wt% of cement.

^b Per unit of mass = 1% by weight of cement.

TABLE 5.9
Structural Elements Based on Environmental Conditions

Element type	Degree of exposed tension surface to environmental factors
Tension surface protected	All internal members in building The element is always immersed in water without any aggressive materials The last floor that isolated against humidity and rain
Tension surface not protected	All structure direct to weather as bridges and last floor not isolated Structure’s tension surface is protected but near coast Element exposed to humidity as cannot be away as open hall or parking
Tension surface exposed to aggressive environment	Element exposed to higher relative humidity Element exposed periodically to relative humidity Water tanks Structure exposed to vapors, gases, or chemicals with high effect
Tension surface exposed to oxides causing corrosion	Element exposed to chemical vapor causes corrosion Other tanks, sewage, and structure exposed to seawater

TABLE 5.10
Minimum Concrete Cover Thickness

Type of element	Concrete cover thickness (mm)			
	For all elements except slab		Walls and solid slab	
	$f_{cu} \leq 250$	$f_{cu} > 250$	$f_{cu} \leq 250$	$f_{cu} > 250$
Tension surface protected	20	15	15	10
Tension surface not protected	25	20	20	15
Tension surface exposed to aggressive environment	30	25	25	20
Tension surface exposed to oxides causing corrosion	40	35	35	30

5.3.6 EXECUTING CONCRETE COVER

From the previous sections, the importance of concrete cover has become clear. Several practical methods are used to make sure the thickness of concrete cover matches design requirements during the construction process. The most famous method uses a piece of concrete cube, called a “biscuit,” of about 50 × 100 mm; the thickness depends on the required cover thickness, as shown in the photo in figure 5.1. During pouring, a steel wire is put into this concrete cube and the steel bars are tied with these pieces to maintain the spacing between the bars.

The disadvantage of this method is that moving the workers on the steel bars to pour concrete or to do any other activities, such as inspection or supervision, exerts a concentrated pressure on concrete pieces and can lead to cracks and damage.

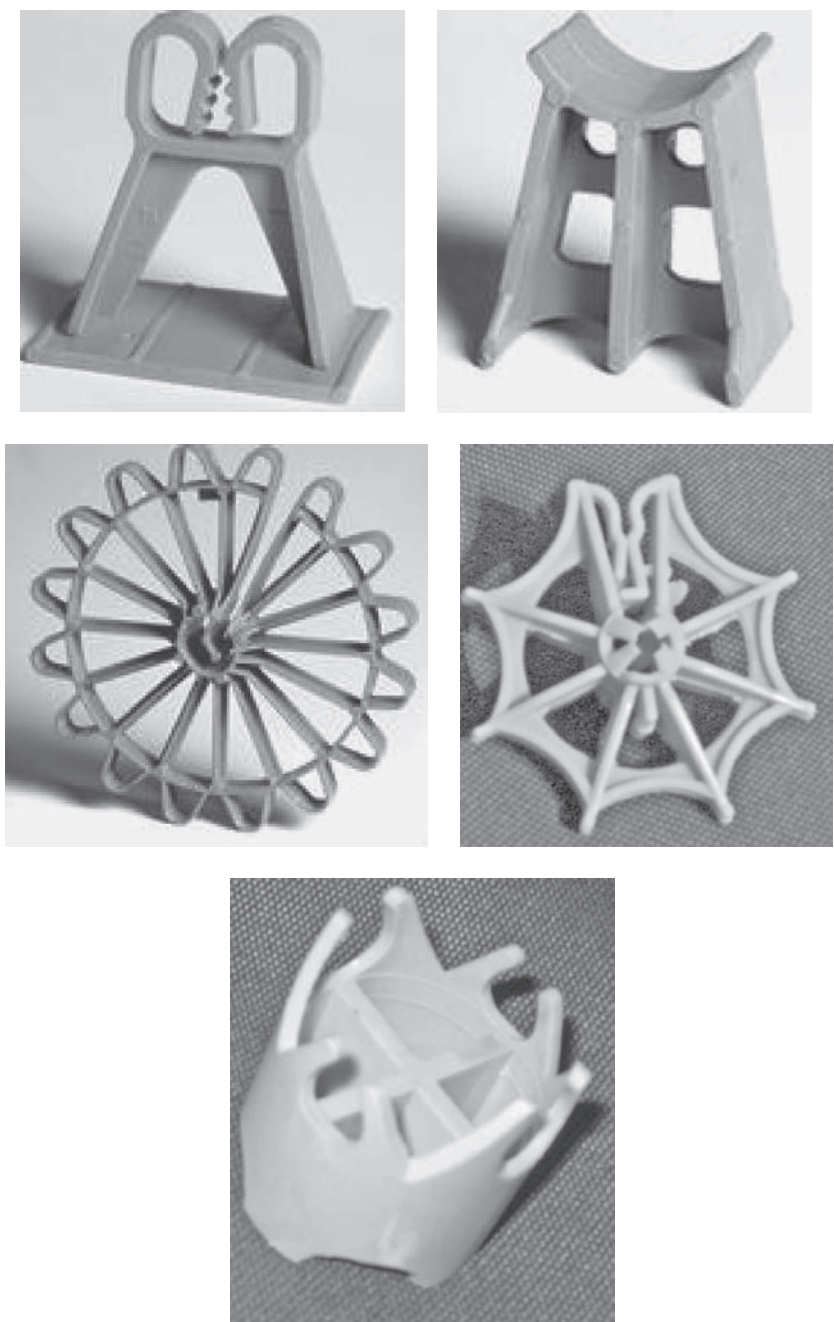


FIGURE 5.1 Photos of different kinds of plastic pieces.

Therefore, after construction, in many cases the cover thickness will be found to be very small or will vanish. The advantage of this method is that it is considered very inexpensive because the poured concrete is the same as that used on-site and the workers are available on-site, also.

A more practical method is to use a plastic piece that will maintain the concrete cover; it will cost little money, and it verifies that the thickness of concrete cover is as seen in the drawings of the designer. Recently, many contractors have been using these plastic parts, which are cheap and strong, to maintain the cover with high accuracy. The photos in figures 5.1 and 5.2 show different types of plastic pieces based on the bar diameter and the location of the steel bars. These plastic forms are used to stabilize the steel bars, thus maintaining the thickness of the concrete cover along the structural member during construction as slab, beams, columns, or foundations.

Figure 5.1 shows the different types and forms of plastic cover, which vary according to size and form of bars and concrete cover thickness to be preserved. Therefore, the customer must give the company's suppliers all the required information on the quantity and thickness of the concrete cover and diameters of the steel bars to be used, as well as the type of concrete member that will be installed. There are different types of these pieces of plastic, depending on whether they are used in a column, slab, or beam. Recalling the picture of the biscuit form, it is worth mentioning that we have seen that this kind of "biscuit" can fix the steel bars and prevent them from movement, which also ensures that the distance between steel bars is strictly enforced. Figure 5.2 shows the plastic pieces carrying the chair; this is also useful for protection of any piece of steel to be exposed to the concrete's outer surface.

For more than 30 years another method was used to keep the concrete cover from corrosion by putting a reasonable aggregate underneath the steel bars; however, this

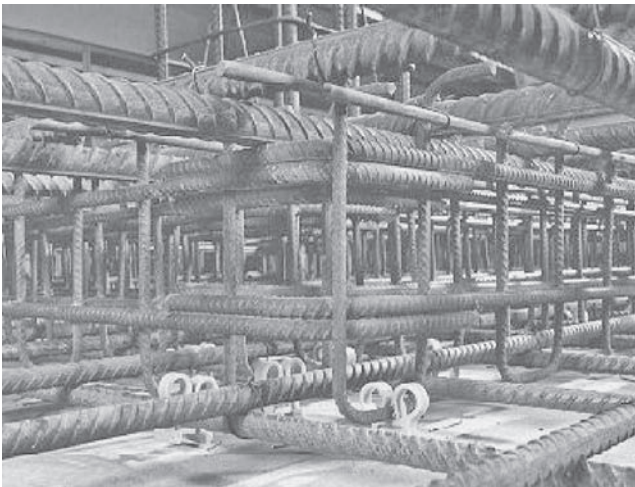


FIGURE 5.2 Plastic pieces carrying chairs.

is now considered a very bad method and is prevented. Nevertheless, this method is still used in some low-cost residential buildings in developing countries because it is very inexpensive. However, future expense occurs due to deterioration of the building and the necessity of performing repairs.

After construction, it is recommended that specifications highlight the importance of measuring the concrete thickness for some special structures exposed to severe conditions. The British standard requires that the measurement of concrete cover be carried out in accordance with BS 1881 Part 204, using an electromagnetic device that estimates position depth and size of the reinforcement. The engineer advises on locations for checking cover and the spacing between measurements based on the objective of the investigation.

Reinforcement should be secured against displacement outside the specified limits unless specified otherwise. The actual concrete cover should be not less than the required nominal cover minus 5 mm. Where reinforcement is located in relation to only one face of a member (e.g., a straight bar in a slab), the actual concrete cover should be not more than the required nominal cover plus:

- 5 mm on bars up to and including 12 mm in size
- 10 mm on bars over 12 and up to and including 25 mm in size
- 15 mm on bars over 25 mm in size

Nominal cover should be specified for all steel reinforcement, including links. Spacers between the links (or the bars where no links exist) and the formwork should be of the same nominal size as the nominal cover. Spacers, chairs, and other supports detailed on drawings, together with such other supports as may be necessary, should be used to maintain the specified nominal cover to the steel reinforcement. Spacers and chairs should be placed in accordance with the requirements of BS 7973-2 and should conform to BS 7973-1. Concrete spacer blocks made on the construction site should not be used.

The position of reinforcement should be checked before and during concreting; particular attention should be directed to ensuring that the nominal cover is maintained within the given limits, especially in the case of cantilever sections. The importance of cover in relation to durability justifies the regular use of a cover meter to check the position of the reinforcement in the hardened concrete. Figure 5.3 shows the electromagnetic cover meter equipment, which is discussed in detail in chapter 4.



FIGURE 5.3 Electromagnetic cover meter.

5.4 MAXIMUM CRACK WIDTH

Cracks in the concrete cover have a direct impact on the process of corrosion in the steel because the presence of cracks assists in the spread of carbon dioxide or chloride ions within the concrete. This starts corrosion and is also the main reason for the entry of oxygen, thereby accelerating the process of corrosion and increasing the corrosion rate. Therefore, the ACI Committee 224 set a maximum permitted width of cracks according to the environmental conditions surrounding the structure, as shown in table 5.11.

When a concrete section is designed to resist corrosion, there are specifications that give the equations of the expected crack width due to load and stresses on the structure. The ACI 224 Committee proposed the following equation to calculate maximum crack width:

$$W_{\max} = 0.076\beta f_s (d_c A)^{0.333} 10^{-3}$$

(5.1)

where

- W_{\max} = the crack width in inches
- β = the ratio of the distances to the neutral axis from the extreme tension fiber and from the centroid of the reinforcement
- f_s = calculated stress (ksi) in reinforcement at service load computed as the unfactored moment divided by the product of steel area and internal moment arm
- d_c = the concrete cover in inches
- A = the area of concrete symmetrical with the reinforcing steel divided by the number of the bars

According to clause 10.6.4 of the ACI code, satisfactory flexural cracks in beams and one-way slabs are achieved if the factor z , defined in equation (5.3), does not exceed the following specified values. In arriving at these values, maximum crack widths of 0.33 and 0.41 mm for exterior and interior exposures, respectively, have been stipulated:

For exterior exposure, $z < 25.4 \text{ kN/mm}$

(5.2a)

For interior exposure, $z < 30.6 \text{ kN/mm}$

(5.2b)

TABLE 5.11
Relation of Environmental Condition and Allowable Maximum Crack Width

Environmental condition	Maximum crack width (mm)
Dry air or protected surface	0.4
High humidity, water vapor, and soil	0.3
Seawater or sprayed seawater	0.15
Barrier walls to water except pipes under no pressure	0.1

This requirement should be satisfied at sections of maximum positive and maximum negative bending moments and is only necessary if the steel used has a yield stress $f_y > 280 \text{ N/mm}^2$:

$$z = f_s \sqrt[3]{d_c A} \quad (5.3)$$

where

f_s = calculated stress (N/mm^2) in the reinforcement bars at service loads, which may be determined by dividing the applied bending moment by the product of the steel area and the internal moment arm or may be considered $0.6 f_y$

d_c = thickness of concrete cover (mm) measured from the extreme tension fiber to the center of the closest reinforcement bars

A = effective tension area of concrete (mm^2) surrounding the flexural tension reinforcement and having the same centroid as the area of reinforcement, divided by the number of bars

If different bar sizes are used, the number of bars to be considered is obtained by dividing the total area of reinforcement by the area of the largest bars.

5.4.1 RECOMMENDED REINFORCEMENT DETAILS OF CRACK CONTROL

Studying equation (5.3) reveals that the following reinforcement detailing assists in reducing the value of z and hence leads to the development of narrower cracks:

- use of the thinnest concrete cover for reinforcement bars consistent with the steel protection requirement and exposure conditions
- in construction, procedure and design detailing, taking into consideration placement of larger size bars in the row closest to the extreme tension fiber whenever more than a single row of reinforcement and different bar sizes are used
- preferably using relatively larger numbers of small bars' diameters rather than a small number of larger bars' diameters, given the same area of steel
- use of least practical distance between rows of bars (To allow the concrete to fill the forms, this distance should not be less than one bar size or 25 mm.)

Note that the cracks occur not only due to loads, but also as a result of exposing the structure to the cycles of freezing and drying that cause plastic cracks or shrinkage cracks or that may occur as a result of some alkaline element in the aggregate. The role of cracks in the corrosion of reinforcement is controversial. Much research shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. For this reason, the former distinction between interior and exterior exposure has been eliminated. These factors can be ignored by taking care about the components of concrete and also in the design of concrete mix, although it is difficult to forecast the crack width

in such cases. In calculating crack width due to loads, the crack's width is impressive, as well as the crack's direction.

Cracks in the direction of the main steel tend to be more influential than in the case of direction with secondary steel, where most steel will be an exhibition of corrosion. Beeby (1979) studied cracks and found that they are often in the direction of secondary steel. He notes that secondary steel does not cause important changes even in the secondary steel in that it does not bear any loads, which would make it dangerous for the safety of structure, and does not play a key role in carrying loads. However, Beeby has also mentioned that if the cover of the concrete falls as a result of the corrosion, the main steel will be corroded quickly; therefore, it is important that secondary steel is protected and also that the stirrups receive the same protection as the main steel does.

5.5 DESIGN PRECAUTIONS IN CASES OF CARBONATION

The carbonation process of concrete can be measured by calculating rates of the spread of carbon dioxide in the concrete, as explained in chapter 3. These equations can be used to anticipate the time required to reach carbonation to steel. Time, t_i , is the time required until the beginning of corrosion. In addition, time, t_p , is the time required for the propagation of corrosion in the steel prior to the spall of the concrete cover. Estimating the value $t_i + t_p$ is the basis of design to control the carbonation process, which causes corrosion.

Parrot (1994) conducted a great deal of research that showed the t_i and t_p clearly. Through some of the factors that are calculated in practice, including factors k , n , and often factor n (equal to 2), the coefficient k is changed, depending on the type of concrete and environmental factors affecting the exposed structure. The value of coefficient k can be identified through practice tests in normal weather. Some experiments were carried out in 1990 by Bentur and Jaegermann, who calculated through the t_i :

$$t_i = \left(\frac{d_c}{K} \right)^{0.5} \quad (5.4)$$

where K is the factor for certain environmental factors and concrete with specific quality, and d_c is the thickness of the concrete cover.

With respect to the factor t_i , Parrot (1994) derived equations based on the following premises:

- The CO₂ factor is the diffusion of air into the concrete cover.
- Spread of CO₂ in concrete depends on CaO content.
- Change of CO₂ content in air will be neglected.
- In cases of higher humidity, the $n = 2$ factor will be different than in the previous equation.

From these assumptions, Parrot proposed the following equation:

$$D = aK^{0.4}t_i^n / c^{0.5} \quad (5.5)$$

where D is a depth of the transformation of carbon and K is the permeability of air (in units of 10^{-16} m^2) of the concrete cover. This value depends on relative humidity and can be estimated from the coefficient value of K at 60% relative humidity in $K = m K_{60}$. The values of m with a different relative humidity are in table 5.12.

- The factor n represents the value of the higher power, always takes a value equal to 0.52, and decreases with increase in the relative humidity almost more than 70% (table 5.12).
- Factor c is the content of CaO in the concrete cover. This factor depends on the local production of cement (as in table 5.12) because it will have different values depending on relative humidity and the types of cement used in Europe.
- The factor a is usually equal to 64, based on experimental results.
- The factor t_p is present at the time of corrosion propagation on the steel and is calculated based on Parrot (1994) in the following equation:

$$t_p = CD / CR \quad (5.6)$$

where CD is the corrosion depth in the steel reinforcement that causes cracks in concrete cover and CR is the steel corrosion rate in the propagation period.

The estimated value of CD by 0.1 mm has been explained before. The CR could account for value according to the relative humidity, as can be seen from table 5.12. These approximate values have been identified based on information and process greatly affected by any external factors. The values in the table may vary by about 10%.

TABLE 5.12
Values m , n , Cement Content (c),^a CR ,^b and t_p ^c

Relative humidity (%)	40	50	60	70	80	90	95	98	100
m	1.00	1.00	1.00	0.797	0.564	0.301	0.160	0.071	0.010
n	0.48	0.512	0.512	0.480	0.415	0.317	0.256	0.216	0.187
c (CEM1b)	460	460	460	460	485	535	570	595	610
c (CEM2b)	360	360	360	360	380	420	445	465	480
c (CEM3b)	340	340	340	340	355	395	420	440	450
c (CEM4b)	230	230	230	230	240	265	285	295	305
CR	0.3	0.3	0.3	2	5	10	20	50	10
t_p	330	330	330	50	20	10	5	2	10

^a Kilograms per cubic meter.

^b Micrometers per year.

^c Year.

Based on the previous equation, the relation between the concrete cover thickness and the structure's lifetime ($t = t_i + t_p$) can be calculated as follows:

$$D = 64K^{0.4} \left[t - (100 / CR) \right]^n / C^{0.5} \quad (5.7)$$

Using this equation, by default, Parrot calculated the relationship between permeability of air and efficiency of the concrete strength by the quality of concrete (concrete strength and curing) and the thickness of the concrete cover to determine true lifetime of structure.

Considering the relationship between concrete strength after 28 days and concrete strength with time f_t after curing more than 28 days, the air permeability of K60 can be calculated by the following equations:

$$f_t = f_{28} \left[0.25 + 0.225 \log(t) \right] \quad (5.8)$$

$$\log(K60) = (30 - f_t) / 10 \quad (5.9)$$

t is the curing time plus 8 days.

The results of the calculations that have been inferred from the previous equation are based on a structure lifetime of 75 years. The required quality of concrete, as well as cover thickness of concrete, is calculated according to the different weather conditions in which the structure exists (see table 5.13).

5.6 DESIGN PRECAUTIONS FOR CHLORIDE EFFECTS

This section will clarify the impact of the design on the concrete mix and the spread of chlorides into the concrete with regard to weather factors. Chloride distribution will be studied to determine the lifetime of a structure under the influence of chlorides. Various atmospheric factors affect concrete structures. For example, although bridges and parking garages are exposed to air, concrete pilings are exposed to seawater. Therefore, the location of a structure is very important because of the weather factors affecting it.

We notice that the lower requirements of the various codes are not enough to preserve the life of a structure; however, it can be maintained to be impermeable concrete with the addition of some ways to protect against corrosion. The basis of the design depends on the nature of chlorides in different times, and the expense of the lifetime can be explained in a simple way by the relationship between the concentration of chlorides and depth at a specific time, as in figure 5.4.

Three main parameters affect the calculation:

1. The effect of the diffusion factor, D_{eff} , depends mainly on concrete quality.
2. The effect of chloride concentration on surface and CO_2 is mainly affected by the weather conditions surrounding the structure and can also be affected slightly by the cement content in concrete.

TABLE 5.13
Required Concrete Quality under Different Weather Conditions^a at Lifetime of 75 Years

Weather condition	Cover thickness	Minimum cover thickness	K_{60} (10^{-16} m^2)	K (10^{-16} m^2)	Concrete cube strength (Mpa)	w/c ratio	Relative humidity
Dry	25	15	36	42	16	0.82	50
	25	15			20	0.90	
Moderate	25	15	46	42	15	0.85	65
	25	15	1.1	0.84	33	0.52	70
	25	15			25	0.65	
Dry/wet	30	20	1.7	1.2	31	0.55	50–100
	30	20	2.5	1.4	29	0.57	60–100
	30	20			35	0.55	
Wet	30	20	13	3.8	21	0.70	90
	30	20	46	7.4	15	0.85	95
	30	20			30	0.60	

^a Calculated from equation (5.3).

- 3. The chloride concentration that causes corrosion needs to be limited. There are two values to start corrosion. The first value is required to start corrosion (C_i) and the second value is required to spall the concrete cover (C_p).

To provide good design and concrete quality, these three factors have been calculated by the approximations of Bamforth (1994) and Berke and Hicks (1992). This will be explained in the following sections.

5.6.1 EFFECT OF THE DIFFUSION FACTOR ON CHLORIDE PERMEABILITY

There are several methods to calculate the diffusion coefficient, depending on what type of concrete is used or, in cases of an unknown type of concrete, nonavailability of data. In the absence of previous concrete data or a time by which to define the chloride content, it is possible to calculate the coefficient by measuring the resistance of concrete to electric conductivity. This is appropriate since the relationship between the resistance of electricity, written with the value of a coulomb (C) and increased chloride permeability test (ASTM C 1204094), works with the many tests. Through data analysis, the value of D_{eff} is obtained through the following two equations:

$$D_{eff} = 54.6 \times 10^{-8} \text{ (resistivity)}^{-1.01} \qquad r^2 = 0.95$$

$$D_{eff} = 0.0103 \times 10^{-8} \text{ (rapid permeability – coulomb)}^{-0.84} \qquad r^2 = 0.97$$

The resistivity is measured by units of $k\Omega.cm$ and permeability is measured by units in coulombs. However, D_{eff} is measured by units of square centimeters per second (cm^2/sec). We can note from table 6.1 in chapter 6 that the temperature factor affects the spread of chlorides and therefore the temperature must be taken into account; this was not taken into account in the previous equations.

In the absence of any concrete data to be used, there is some general information that could be indicative of the value of D_{eff} through a combination of information available from Bamforth (1994) and Berke and Hicks (1992) because they relied on D_{eff} , which directly affected the quality of concrete. Bamforth provided a direct relationship between the diffusion factor and concrete compressive strength, as shown in table 5.8, whereas Berke and Hicks clarified the relationship through the proportion

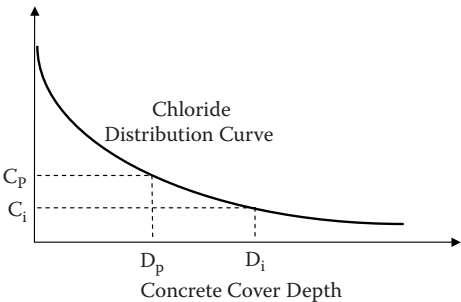


FIGURE 5.4 Sketch represents the relation between chloride concentration and depth at time, T .

TABLE 5.14

Estimate Diffusion Coefficient Values, D_{eff} , as Function of Concrete Mix and Temperature Degree

Mix	D_{eff} (m ² /sec)			
	22°C	10°C	18°C	27°C
Water-to-cement ratio = 0.5	12×10^{-12}	5.3×10^{-12}	9.2×10^{-12}	17×10^{-12}
Water-to-cement ratio = 0.45	6×10^{-12}	2.6×10^{-12}	4.6×10^{-12}	8.3×10^{-12}
Water-to-cement ratio = 0.4	3×10^{-12}	1.3×10^{-12}	2.3×10^{-12}	4.2×10^{-12}
Water-to-cement ratio = 0.35 or 0.4 with fly ash	2×10^{-12}	8.8×10^{-13}	1.5×10^{-12}	2.8×10^{-12}
Equal to 1,500 C	2.6×10^{-12}	1.2×10^{-12}	2.0×10^{-12}	3.4×10^{-12}
Equal to 1,000 C	1.9×10^{-12}	8.3×10^{-13}	1.4×10^{-12}	2.6×10^{-12}
Equal to 600 C	1.3×10^{-12}	5.7×10^{-13}	9.9×10^{-13}	1.8×10^{-12}
Equal to 300 C	0.75×10^{-12}	3.3×10^{-13}	5.7×10^{-13}	1.04×10^{-12}

of water to cement (table 5.14). From table 5.14, the values of the diffusion factor at different temperatures covering warm and cold weather are approximate.

By studying table 5.14, we will find that an increase of temperature increases permeability of chlorides. Therefore, in the case of designing a reinforced concrete structure in warm air, it is necessary to make the concrete with very low permeability. It has been found that in concrete cast from ordinary portland cement, only the permeability coefficient is affected significantly with structural age. However, in the case of concrete to which metallic additives such as silica fume and fly ash have been added, the coefficient of diffusion is influenced mainly by w/c ratio, as well as strength—especially under the influence of weather factors for a long period of time.

5.6.2 EFFECT OF CHLORIDE CONCENTRATION ON SURFACE

Chloride concentration on the surface is directly affected by weather factors and the value of chloride concentration on the surface is measured in units of weight and percentage units or kilograms per cubic meter (kg/m³) of concrete. Berke and Hicks (1992) emphasize that these values function in the weather factors directly; various field studies have used the following values in design considerations:

- Offshore structures: Chlorides speed up to a fixed value of 18.7 kg/m³.
- Structures prone to saline air: Chlorides are an average increase of 0.15 kg/m³/year to the maximum value of 15 kg/m³/year.

Bamforth (1994) studied the impact of concrete components on chloride concentration on the surface. He found that with the increase in the cement content, the percentage of chlorides on the surface will increase. If the cement content increases from 200–400 kg/m³, the chloride content will be 0.5–0.6%, with about 20% increase. However, when the content of cement increases from 400–600 kg/m³, chloride content increases 0.6–1.2% at 100%. Thus, chloride content on the surface must be taken into account in cases of cement content equal to or more than 400 kg/m³.

5.6.3 EFFECTIVE CHLORIDE CONCENTRATION

The concept of the initial concentration of chlorides affecting corrosion has been studied. The value of chloride concentration is necessary for the beginning of corrosion or for breaking a passive protection layer. In laboratory analysis and in the field, it has been proposed that the time needed for the spread of corrosion in steel until the concrete cover cracks and falls is based on specific content per unit of mass, C_p . C_i and C_p are units as a percentage of the weight of cement, or kg/m³ of concrete. Bamforth (1994) proposed these values:

$$C_i = 0.4\%$$
$$C_i = 0.9 \text{ kg/m}^3$$

$$C_p = 1.5\% \text{ of cement weight}$$
$$C_p = 3 \text{ kg/m}^3$$

From a scientific point of view, these are approximately the same values. In the previous case of the concrete using cement content of 300 kg/m³, the suggested values by Bamforth are $C_i = 1.2 \text{ kg/m}^3$ and $C_p = 3 \text{ kg/m}^3$. These are considered the same values as those suggested by Berke and Hicks (1992).

5.6.4 CALCULATING STRUCTURE LIFETIME

From the previous equations, one can calculate structure lifetime or determine the required thickness of the concrete cover, as well as the quality of concrete suitable for life expectancy, on the basis of design. Bamforth used model and laboratory data for the calculation of life expectancy by British code, which is exposed to atmospheric factors in table 4.8. The lifetime of a structure in the case ($C_p = 1\%$) is often less than 20 years.

The required quality of concrete expressed by D_{eff} and the thickness of the concrete cover for a structure lifetime of 50 years (or 120 years in the case of exposure to chloride) as calculated by Bamforth is shown in table 5.15. The calculation is based

TABLE 5.15
Concrete Cover Thickness to Achieve 50- or 120-Year Lifetime for Building in a Chloride Environment

Maximum diffusion coefficient				Concrete cover (mm)
$C_p = 1\%$		$C_i = 0.4\%$		
Age = 120 years	Age = 50 years	Age = 120 years	Age = 50 years	
9.06×10^{-14}	2.17×10^{-13}	4.42×10^{-14}	1.06×10^{-13}	30
1.61×10^{-13}	3.87×10^{-13}	7.86×10^{-14}	1.89×10^{-13}	40
2.52×10^{-13}	6.04×10^{-13}	1.23×10^{-13}	2.95×10^{-13}	50
4.25×10^{-13}	1.02×10^{-12}	2.07×10^{-13}	4.98×10^{-13}	65
5.66×10^{-13}	1.36×10^{-12}	2.76×10^{-13}	6.63×10^{-13}	75
8.16×10^{-13}	1.96×10^{-12}	3.98×10^{-13}	9.54×10^{-13}	90
1.01×10^{-12}	2.42×10^{-12}	4.91×10^{-13}	1.18×10^{-12}	100

TABLE 5.16
Concrete Mix Limit^a

	No risk from corrosion	Corrosion due to carbonation				Corrosion due to seawater chloride			Corrosion due to chloride without seawater			Corrosion due to chemical attack		
		XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XA1	XA2	XA3
Maximum w/c ratio		0.65	0.6	0.55	0.5	0.5	0.45	0.45	0.55	0.55	0.45	0.55	0.5	0.45
Concrete compressive strength	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45
Minimum cement content (kg/m ³)		260	280	280	300	300	320	340	300	300	320	300	320	360

^a Based on ENV 206.

on the beginning of corrosion C_i at 4% of the weight of cement and cracked concrete at the concentration of $C_p = 1\%$ of the weight of the cement. To get the thickness of concrete cover less than 50 mm, the D_{eff} must be less than 10^{-13} ; that can be obtained with a high quality of concrete with additives.

The value of the concrete diffusion coefficient, D_{eff} , for concrete with w/c ratio is equal to 0.45%, required by the code. In the case of a structure affected by bad weather and the increasing concentration of chlorides, it did not satisfy the requirements in the code as the previous one with 50–75 mm thickness of the concrete cover.

5.6.5 GENERAL DESIGN CONSIDERATIONS

It is clear that when any structure is designed, it is necessary to identify accurately the concrete quality of the structure and the degree to which it is exposed to the weather factors affecting it over its lifetime; these cause steel corrosion and therefore determine the thickness of the concrete cover of each member on the structure. Note that the Egyptian code has carefully defined various weather factors that affect concrete structures (also see table 5.9). The code of the European Union has identified the weather factors to which structures are exposed (table 5.16). A set of specifications, European ENV 206 (1992), set final revisions in April 1997 (Bentur et al.) and are presented here only to clarify how best to design buildings to achieve concrete with a longer lifetime.

In determining the form of structures, it is necessary to find out the structure of the concrete, according to given value characteristics of the concrete. The required percentage of water to cement must not exceed it, and the content of cement must not transfer it (see table 5.17). Therefore, generally, to achieve the longest possible building lifetime, one must determine the thickness of the concrete cover, concrete characteristic compressive strength, and cement content in concrete, as well as control crack width to be consistent with the code followed in the design.

TABLE 5.17
Different Degrees of Effect for Different Environmental Conditions

Deterioration type	Exposed degree	Surrounding environment	Example
No possibility for corrosion	XO	No effect on concrete	Concrete inside the building with very little relative humidity
Corrosion due to carbonation	XC1	Dry	Concrete inside the building with very little relative humidity
	XC2	Wet and rarely dry	Some foundations or parts from retaining wall to water
	XC3	Moderate relative humidity	Concrete inside building with moderate relative humidity or outside protected concrete from rain
	XC4	Cycles of dry and wet	Exposed to water, not like XC2
Corrosion due to chlorides in the sea	XS1	Air with salts but not on coast	Building near the coast
	XS2	Immersed	Part of offshore structure

(continued)

TABLE 5.17**Different Degrees of Effect for Different Environmental Conditions (continued)**

Deterioration type	Exposed degree	Surrounding environment	Example
Corrosion due to chlorides	XS3	Splash zone	Part of offshore structure
	XD1	Moderate relative humidity	Concrete surface exposed to water spray containing chloride
	XD2	Wet and rarely dry	Swimming pool or concrete exposed to water from industrial waste
Freezing and melting	XD3	Cycles of dry and wet	Parts of bridges
	XF1	Moderate saturation without ice melting	Main surface exposed to rain
	XF2	Moderate saturation with ice melting	Main surface in road
	XF3	High water saturation without melting ice	Horizontal surface exposed to ice
	XF4	High water saturation with melting ice	Roads, bridges, deck, or main surface exposed to melting ice
Chemical attack	XA1	Exposed to slight chloride attack	
	XA2	Exposed to moderate chloride attack	
	XA3	Exposed to high chloride attack	

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6 Controlling Corrosion in Steel Bars

6.1 INTRODUCTION

Controlling the corrosion process in reinforcement steel bars buried in concrete is accomplished by controlling as much as possible all the factors that help to start the corrosion and spread it throughout the steel reinforcements in the concrete structure. All codes contain different requirements and specifications concerning the required thickness of the concrete cover, the water-to-cement (w/c) ratio in the mixture, and cement content based on the weather factors surrounding the structure. These requirements have been included in the code after many field and laboratory tests have been performed in order to reach specific guidelines and recommendations.

To protect concrete structures from corrosion, one must bear in mind some precautions for use of concrete specifications commensurate with the weather factors surrounding the structure. These will be listed in this chapter, where we will discuss how to control the corrosion process. As for corrosion protection for the structure, several methods and new advanced techniques will be clarified in detail in chapter 7. These protection methods and techniques do not dismiss the need to control the corrosion process through using good quality-control procedures in concrete construction.

Different codes and specifications provide us the requirements and limits to be followed in the design and implementation of concrete structures, such as the thickness of the concrete cover and the proportion of chlorides in the mix, as explained in chapter 5. However, this does not obviate the use of some other precautions during these phases of the concrete mixing. For example, some countries have weather conditions that result in special economic circumstances, necessitating the development of many precautions during construction. Therefore, this chapter will explain how to control the process of corrosion by controlling the process of building the concrete structure. It notes some simple ways of taking precautions during construction at a lower cost compared to other simple ways of protection, which will be explained in chapter 7. The cost of these requirements is an especially low cost that is not comparable to the high cost of the future's repair processes.

It is necessary to control corrosion by taking into account the requirements of the construction; a suitable design will consider very good protection without adding any extra cost to the project. The control of the corrosion process depends mainly on controlling the two factors causing the corrosion: the carbonation transformation process and spread of chloride into concrete. The control of these two operations occurs in three main ways: study of the impact of weather factors on the process of corrosion, the component of the concrete mix and its impact on the process of corrosion, and the curing process for concrete. Understanding these processes and

their effects on the process of corrosion from carbonation transformation or chloride propagation will provide the ability to choose the required method of curing and appropriate mixing ratios to control corrosion as much as possible.

6.2 CARBONATION PROCESS CONTROL

The carbonation process was clarified in chapter 4 and it can be defined simply as the propagation of carbon dioxide into the concrete until the CO_2 reaches the steel bars. This process then breaks the passive protection layer around the steel bars and, consequently, the corrosion process starts. The carbonation process is highly affected by the weather conditions.

6.2.1 EFFECT OF ENVIRONMENTAL CONDITIONS

The carbonation transformation process is affected greatly by the amount of moisture in concrete. This is very clear in figures 6.1a and 6.1b and based on a study performed by Wierig in 1984. From these figures, it is found that when relative humidity is very high, the concrete pore voids will be full of water and it is difficult for CO_2 to spread inside the concrete. Conversely, in cases of very low relative humidity, there

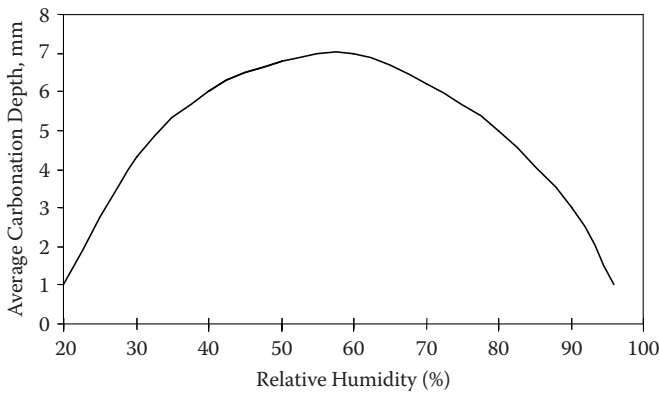


FIGURE 6.1a Effect of relative humidity on carbonation depth at 2 years of age with 0.6 w/c.

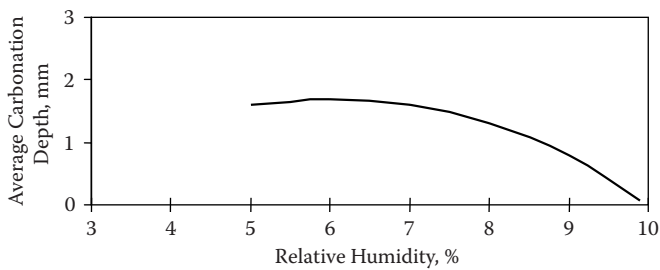


FIGURE 6.1b Effect of relative humidity on carbonation depth at age of 16 years and w/c = 0.6.

will be little moisture, so the carbonation chemical reaction will be difficult and the carbonation transformation process slow. As shown in these two figures, the results are the same when the building’s age is 2 years or 16 years.

We must differentiate between the relative humidity of the air space surrounding the structure and relative humidity inside pore voids of concrete. Also, note that concrete tends to dry slowly with depth and that the moisture varies with the time in the concrete cover. This shows that the rate of carbonation transformation calculation as a function of the relative humidity is not entirely accurate. Our inquiries can be answered, however, because many equations calculate the depth of carbonation and most of them are empirical equations; this was detailed in table 3.1 in chapter 3.

Some other factors also affect the depth of carbonation—for example, when concrete is exposed directly to the rain, or concrete in special circumstances that take the structure away from exposure to rain, or the w/c ratio (fig. 6.2), according to studies performed by Tutti in 1982. From figure 6.2, one can find that the carbonation transformation depth increases with time. In any case, this depth is affected by w/c ratio. After 50 years, the carbonation transformation depth is equal to around 15 mm at w/c ratio equal to 0.45 and reaches over 50 mm when w/c ratio is equal to 0.75.

Concrete under shelter that is protected from rain gives a higher carbonation depth than when it is without shelter and unprotected from rain. Calculating the rate of carbonation within the concrete through practical tests gives results that differ from those from application on-site; thus, in lab tests relative humidity is often between 40 and 60%, which is the region in which the carbonation process occurs quickly, as seen in figure 6.1. Therefore, the laboratory tests give the worst cases of carbonation, as shown in figure 6.3. From this figure, one can find that the relative carbonation depth increases in the laboratory more than in the field test. Also, protected concrete gives higher results than those of unprotected concrete.

From this, one can conclude that the concrete inside a building, which is protected from the outside, is more highly influential to the carbonation process compared with exterior concrete exposed to the outside without protection—assuming that the same source of CO₂ is available in the two cases. However, it is usually the

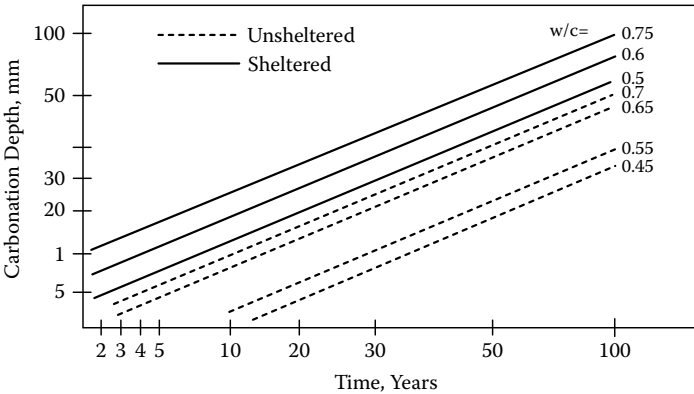


FIGURE 6.2 Relation between carbonation depth and w/c for different conditions.

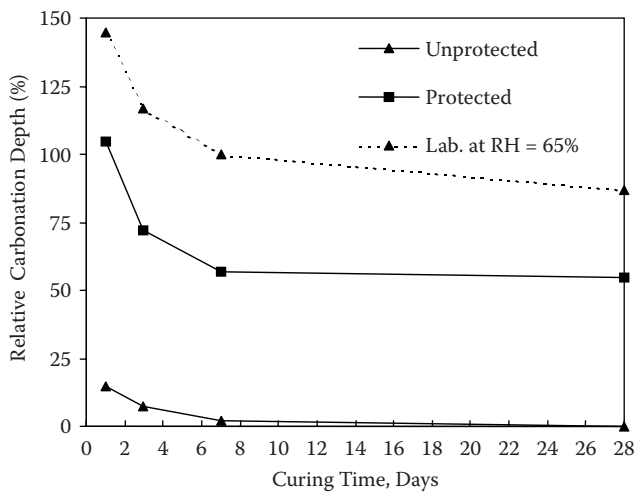


FIGURE 6.3 Curing effect and surrounding environment at carbonation depth of concrete at 16 years of age.

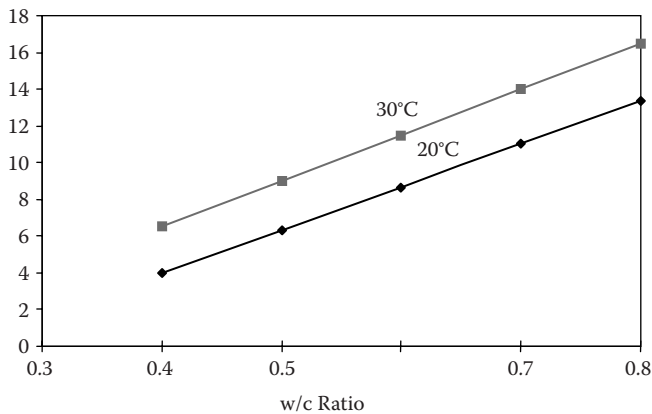


FIGURE 6.4 Effect of temperature and w/c ratio on carbonation depth after 15 months. (From Jaegermann, C. and D. Carmel, 1988).

exterior member that is exposed to higher amounts of CO₂ due to the surrounding environmental conditions.

One of the weather factors that affects the process of carbonation is the temperature. With increasing temperature, the carbonation process is also increased, as shown in figure 6.4. This figure shows that when the temperature is increased from 20–30°C, the carbonation process is increased by 50–100% with a different w/c ratio. These tests are performed by putting the samples in the atmosphere for a period of about 15 months. Note in this figure that the higher w/c ratio will result in increase in the carbonation depth. Also, under any w/c ratio, the depth of carbonation is higher with the increase of air temperature. This result is very important, especially for the

Middle East, where countries are exposed to high temperature with high sources of carbonation, especially in petrochemical plants. Therefore, as we will discuss about maintenance strategy in chapter 9, concrete structures that exist in higher ambient temperatures need less periodic maintenance time.

When developing the building assessment, it is important to know the temperature around the deteriorated concrete member as it can provide a reason for corrosion due to carbonation. Note that, even though when the structure is inspected the ambient temperature is good, the structure may have been exposed to high temperatures over a period of time. Therefore, it is important to collect the data about the ambient temperature affecting the concrete throughout its lifetime because this information could change the choice of repair and future maintenance plans for the building. Also, figure 6.4 shows that at any ambient temperature, the carbonation transformation depth is increased by increasing the w/c ratio.

6.2.2 COMPONENTS OF CONCRETE MIX

Weather factors cannot be controlled by the engineer who does the construction at the site, but he can control the components of the concrete mixture as well as the cement content or the w/c ratio and the process of curing after casting. Generally, the impact of the w/c ratio is very high, as shown in the previous figure. With the increase in the w/c ratio, depth of carbonation increases. On the other hand, the impact of the use of slag furnaces (fly ash) is clear in figure 6.5. As the figure shows, when fly ash is used, the depth of carbonation is higher than when ordinary portland cement is used. In both cases, increasing the w/c ratio will increase the depth of carbonation.

As stated in Hobbs’s (1994) research, with increasing concrete compressive strength, the carbonation depth will decrease. This test is performed at a building age of 8.3 years and is done for concrete containing ordinary portland cement; another test is done when fly ash is used. This relationship has been created through a curve that gives a rapid increase in the depth of the transformation of carbonation with a decrease of concrete compressive strength. Figure 6.6 shows the effect of the depth of carbon transformation by the concrete compressive strength for different ages of 1, 10, and 50 years. With increase of the lifetime of a building, the depth of the carbonation transformation increases. This test was carried out by Smolczyk (1968). It notes that from the first year up to 10 years, the carbonation depth is decreased

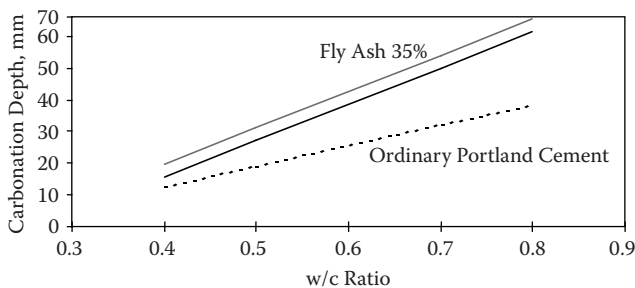


FIGURE 6.5 Relation between w/c ratio and carbonation depth after 100 months.

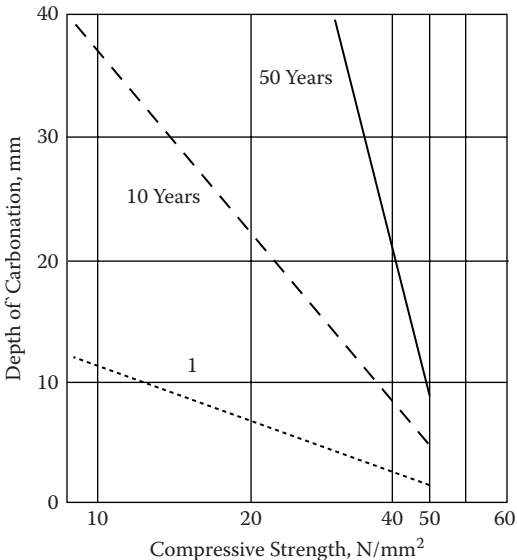


FIGURE 6.6 Effect of concrete compressive strength on carbonation depth at various ages.

with increasing concrete compressive strength, but does not decrease at the same rate at the age of 50 years.

It is important to know from figure 6.6 that at 20 N/mm^2 carbonation transformation depth will be around 24 mm after 10 years, whereas in the same conditions at 40 N/mm^2 , it will be 5 mm at 10 years; thus, the decrease in carbonation depth is about five times higher. Note in the figure that the carbonation depth is 20 mm at 50 years. This information is valuable when preparing a maintenance plan and selecting a suitable concrete compressive strength. When 20- N/mm^2 compressive strength is selected, it will be less expensive than 40 N/mm^2 . On the other hand, with 20 N/mm^2 , the steel bars start corrosion after 10 years; however, for 40 N/mm^2 compressive strength will start the corrosion after 50 years, so no maintenance may be needed for the project’s whole lifetime.

6.2.3 CURING

From the previously mentioned tests, results are measured for curing concrete by water or under laboratory conditions, which are different from the practical conditions found on-site. Curing in laboratory tests for cylinder or cube standard compressive strength is performed by submerging the samples in a sink of water for 28 days; this is completely different from the field. Therefore, the results would be different on-site during construction, when, in practice, curing can be inadequate, especially in areas with warm, dry air.

The effect of curing duration and w/c ratio on the transformation of carbonation is also clarified in figure 6.7. The figure reflects a coastal city in a warm, dry region on the Mediterranean Sea, where temperature is 30°C and relative humidity is 40% after 5 years. Improper curing will increase the depth of the carbon transformation

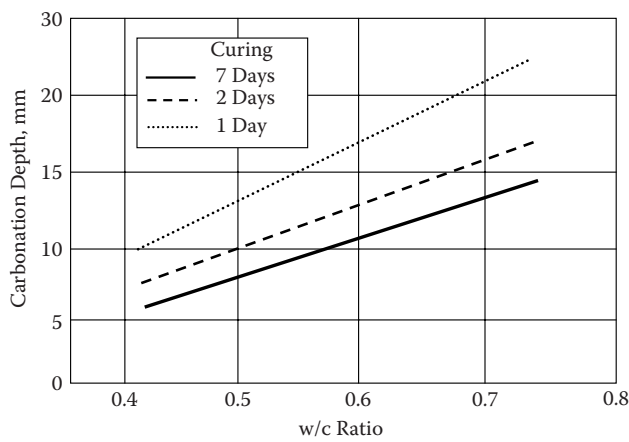


FIGURE 6.7 Effect of w/c ratio and curing on the carbonation depth.

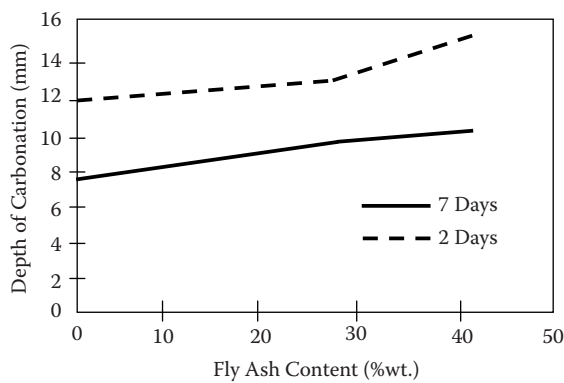


FIGURE 6.8 Effect of curing on carbonation depth at normal conditions.

rate to two to four times that for a proper curing process. Note that the worst conditions do not assist in proper curing when the temperature is high with dry air and also with a moist atmosphere.

Therefore, we must care about concrete mixing ratio and follow up the quality of the execution; consequently, we must care about w/c ratio and the curing process. The curing process is important for two reasons: Improper curing will decrease the concrete compressive strength about 20% (Bentur and Jaegermann 1990) and it will have a significant impact on increasing the depth of carbonation transformation. The impact of improper curing of concrete is greater when fly ash is used in the concrete mix because the reaction is slow, as is obvious in figure 6.8. With the increasing amount of fly ash, more depth of carbon transformation will be found; after 2 days, the rate of increasing carbonation transformation depth will be higher after increasing the percentage of fly ash content by about 30% (fig. 6.8).

The adverse effect of increasing the depth of the carbonation transformation with increase of slag furnace content must be considered when buildings are exposed

to the influence of the carbonation process. Therefore, the designer must think about not using fly ash in concrete if the building will be exposed to carbonation during its lifetime. Note that attention must be given to a curing process using fly ash and that the curing process varies when additives are used because their properties are different even when they are of the same nature (such as silica fume, slag, and fly ash). These differences in their behavior do not exist in the specifications.

Therefore, in the case of mineral additives, the engineer must decide whether they will improve the quality of concrete and must also examine the appropriateness of the nature of the structure and of the work, as well as the circumstances of the weather. It is already clear that inattention to the process of concrete curing after casting has a great impact on concrete compressive strength as well as on the carbonation process. From earlier figures, note that there is little difference in the effect of curing duration on the lack of depth of the carbon transformation. We know that the curing process certainly does not cost much compared to the problems arising from lack of a curing process.

6.3 CHLORIDE CONTROL

Chlorides are the second main cause of corrosion of steel bars embedded in concrete. The steel bars will be affected by the presence of chlorides inside the concrete itself or by chlorides from outside the structure, such as in seawater, seawater spray, or a surrounding atmosphere saturated with salt. The different codes and specifications provide chloride limits in concrete mix and confirm that the content of chlorides in the concrete has no effect on start of corrosion. Moreover, this should reduce the chloride content over the lifetime of the structure.

6.3.1 WEATHER FACTORS AFFECTING CORROSION

Chlorides spread inside concrete as a result of chlorides inside the same concrete during casting or outside the structure due to seawater (e.g., an offshore structure). Chloride propagation inside concrete depends on the equation of Fick’s second law, which was explained before, as well as on the diffusion factor. The natural factors that affect structures have a very significant impact on the probability of causing

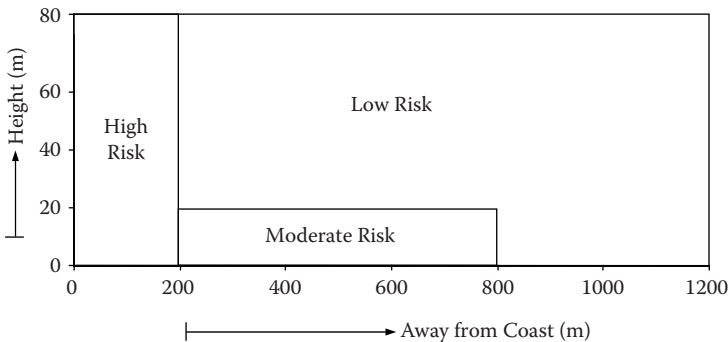


FIGURE 6.9 Structure corrosion risk based on height and distance from the coast.

corrosion to the structure. This serious factor functions in the presence of chlorides in the surrounding environment, which could infiltrate the concrete. They must be given appropriate attention—for example, the splash zone in the case of offshore structures and other structures, such as bridges, where salt is used to melt ice.

Chloride can be in the form of spray in the air when a marine environment surrounds the structure; an obvious example is that of a building under construction near the shore in a coastal city. Figure 6.9 shows the degree of serious threat to a structure according to its location in terms of its distance from the coast and also in terms of height. However, these data are not taken as a whole, but only as a guide to determine the degree of serious threat, depending on the climate conditions of the region.

Another important factor is air temperature, especially in marine areas with warm atmospheres (e.g., the Arabian Gulf and the Middle East). Note the increase of the temperature diffusion coefficient in the following equation:

$$D_2 = D_1(T_1 / T_2)e^{[k(1/T_1 - 1/T_2)]} \tag{6.1}$$

where D_2 and D_1 are the diffusion coefficients at the temperatures T_2 and T_1 , respectively. The results of laboratory tests clarified the relationship between the quality of concrete and increase of the temperature on the diffusion coefficient.

From this relationship and also as presented in table 6.1, note that, with the increase of the temperature coefficient, the diffusion coefficient increases and that the diffusion factor increases about twofold in the event of an increase in temperature of about 10°C. At any temperature, the lower the w/c ratio is, the lower the coefficient of diffusion will be. Also, when silica fume is added to the concrete mix, it will reduce the diffusion factor about half, as shown in the previous table. However, when the temperature is increased from 10–20°C, the diffusion factor is increased about 100%.

6.3.2 COMPOSITION OF CONCRETE MIX

The main factors that control the components of the concrete mix design are the w/c ratio, the cement content, and the coarse aggregate-to-sand ratio. When the impact of chlorides on the corrosion of steel reinforcements is studied, the main factors affected will be the w/c ratio and the cement content. Figure 6.10 shows the relationship of the w/c ratio, as well as the components of the cement, on the time required for the start of corrosion. Rasheduzzafar et al. (1990) conducted experiments in which samples

TABLE 6.1
Effect of Weather Temperature and Concrete Type on the Diffusion Factor

Concrete type	Diffusion factor effect (× 10 ⁻¹² m ² /s)	
	10°C	22°C
Water-to-cement ratio = 0.4	1.3	3
Water-to-cement ratio = 0.3	0.9	2
Water-to-cement ratio = 0.4 and 7.5% silica fume	0.6	1.3

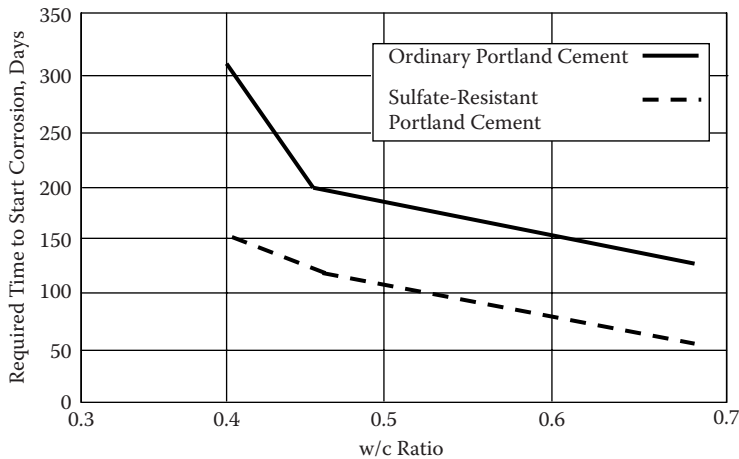


FIGURE 6.10 Effect of C_3A and w/c on the required time to start corrosion.

of concrete were partly submerged in a 5% solution of sodium chloride. From this figure, one can find that, with the increase of w/c ratio, the time for the beginning of corrosion will be decreased. In addition, the use of sulfate-resistant cement reduces the time required for corrosion to begin.

Verbeck (1975) stated that an increase of C_3A content in portland cement will delay the deterioration and fall of concrete due to cracks resulting from corrosion in the steel reinforcement for the structures exposed to seawater. The different codes give the minimum limit of cement content required, but it is obvious that with the increase in cement content, the chloride diffusion factor is decreased at the same w/c ratio, as shown in figure 6.11. When the w/c ratio is equal to 0.4, the effect of increasing the cement content in the concrete mix on the diffusion factor will decrease. Therefore, the cement content and the w/c ratio are the main factors that control the degree of chloride propagation inside the concrete. Figure 6.11 presents the test results of samples exposed to seawater according to research by Pollock in 1985.

6.3.3 CURING FOR CHLORIDE ATTACK

As we stated in the case of the carbonation process, as well as in the case of chlorides, proper curing can have a significant impact, as it affects the spread of chlorides in the concrete, especially in the case of structures exposed to marine environments and warm air. The relationship of the depth of concrete containing 0.4% chloride in the range of 1–5 years and a w/c ratio equal to 0.5 is presented in figure 6.12 (Jaegermann and Carmel 1988, 1991). This figure shows that, in any case, the depth of chloride in a curing process performed in 7 days is less than that in curing for only 2 days; hence, the curing process has a direct impact on the depth of chloride penetration in the concrete. Note also that during the first year, the impact of curing on the depth of propagation of chlorides is higher than that in effect after about 5 years.

When comparing carbonation propagation within concrete or chloride propagation and rear access to depth at 0.4% concentration of chlorides in the same

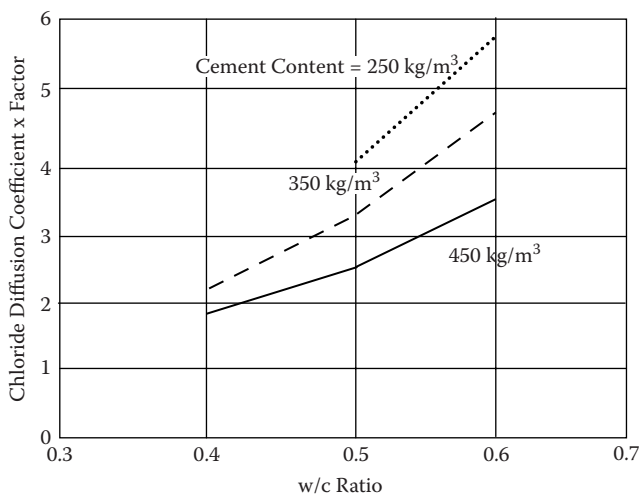


FIGURE 6.11 Effect of w/c and cement content on the chloride propagation factor in concrete exposed to seawater.

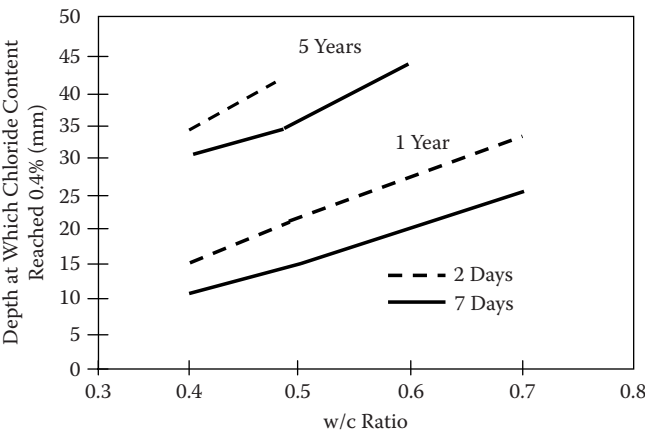


FIGURE 6.12 Effect of curing and w/c ratio at depth with chloride content 0.4% by cement weight in warm weather.

period of curing and with the same w/c ratio, note that the chloride spreads faster than carbonation does. Comparing the effects of using ordinary portland cement or cement containing fly ash with 25% by weight of cement, based on Jaegermann and Carmel (1988), shows that the case performing quick curing and the w/c is equal to 0.5 after 1 year of exposure to warm sea weather. We can note an increasing percentage of the chloride content with depth within the concrete when fly ash is used compared to when ordinary portland cement is used. Therefore, when fly ash additives are used, the engineer must be sure to make the initial tests to determine the required quantity necessary not to have an impact on corrosion control on the steel reinforcement.

6.3.4 EXECUTION OF CURING

Recently poured concrete must be protected from rain and fast drying as a result of severe storms or warm, dry air. This is done by covering the cast concrete by a suitable coating system after finishing the pouring process until the time for final hardening. The curing process then proceeds in a manner already defined by the designer. Figures 6.13, 6.14, and 6.15 illustrate the impact of relative humidity, air temperature, and wind speed on the speed of concrete drying.

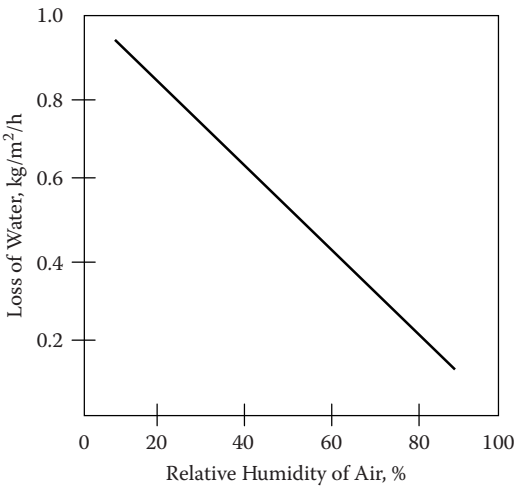


FIGURE 6.13 Relation between relative humidity and loss of water. (From Neville, A. M., *Properties of Concrete*.)

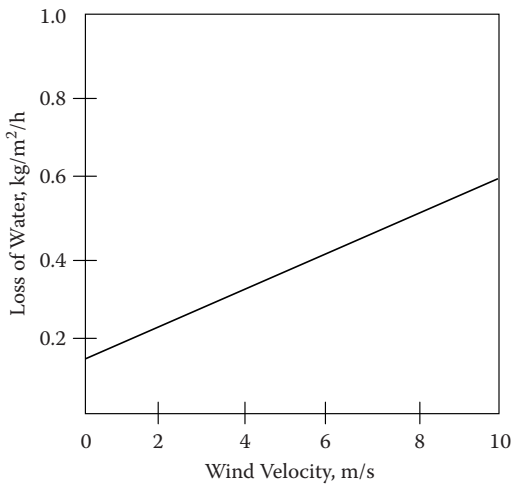


FIGURE 6.14 Relation between wind speed and loss of water. (From Neville, A. M., *Properties of Concrete*.)

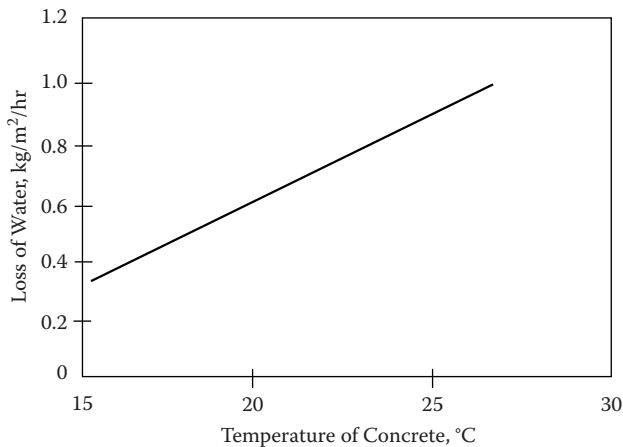


FIGURE 6.15 Relation between temperature and loss of water.

The purpose of the curing process is for the concrete to be wet for a period of not less than 7 days for ordinary portland cement and not less than 4 days in the case of fast hardening or if additives have been used to accelerate the setting time. The curing process can be performed in many ways, including:

- spraying water that is free of salt and any harmful substances
- covering the concrete surface by a rough textile, sand, or manufacturer wood waste and keeping it wet by spraying water on it regularly
- covering the concrete surface by high-density polyethylene sheets
- spraying additives on the concrete surface
- using steam for curing in some special structures

Figures 6.16, 6.17, and 6.18 represent different types of curing. Figure 6.16 illustrates the second common method of curing by covering the slab with a textile that is always kept wet. The first curing method is spraying water on the concrete member in the early morning and at night, avoiding spraying water after sunrise as it will produce some cracks on a concrete surface due to evaporation of water. This method is used for normal environmental conditions. However, in some areas, such as the Middle East, the temperature in summer may reach as high as 55°C. In these conditions, different techniques are used, such as using some chemical to spray on the surface to prevent evaporation of the water in the concrete mix, as shown in figure 6.17. Figure 6.18 shows the other method: covering the concrete member with a special plastic sheet to avoid the evaporation of water.

Another type of method is to spray the remnants of the wood manufacturing process, distribute them on the slab, and keep them wet all the time. A similar idea is to distribute sand on the slab and spray it with water so that it is always wet, taking care that the wet sand has a high dead weight that can affect the building from its recent construction. The curing process involves special issues: It costs little compared to the cost of concrete as a whole; it increases the resistance of concrete in a



FIGURE 6.16 Curing by using textiles.



FIGURE 6.17 Spray chemical.



FIGURE 6.18 Plastic sheet to protect from evaporation.

very significant way when care is taken during the curing process, the strength of the concrete will increase. Figure 6.19 shows that the concrete strength clearly increases when the curing process take 7 days—more than that in the case of 3 days' curing. A

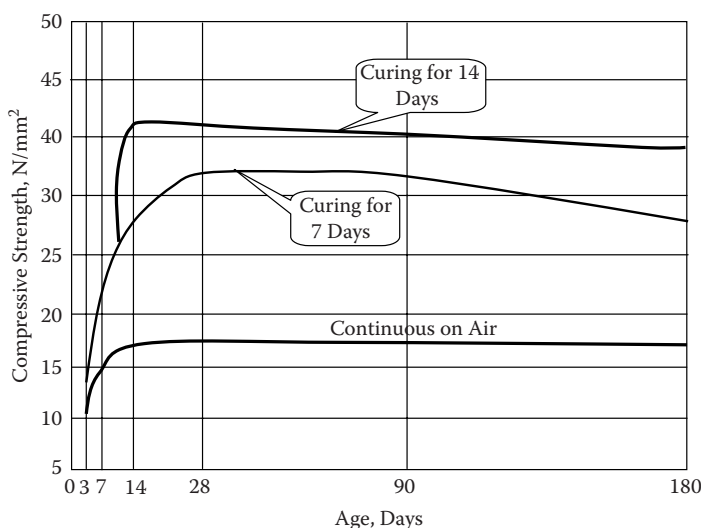


FIGURE 6.19 Sketch presents the relation between curing time and compressive strength.

stronger effect will be found when the duration of curing is 14 days, and this increase in concrete strength will be continuous throughout the structure's lifetime.

Moreover, the increase in concrete strength due to curing for 28 days has no more effect than that from the process of curing for only 14 days. The specifications of the project should provide clearly for a method of curing and define the required curing time exactly as they differ from project to project, according to the weather factors of the area in which the project is located and according to the concrete members.

6.3.4.1 The Curing Process in ACI

According to ACI clause 5.11, the curing of concrete should be maintained above 50°F and in a moist condition for at least the first 7 days after placement; high-early-strength concrete should be maintained above 50°F and in a moist condition for at least the first 3 days. If it is necessary to accelerate curing by high-pressure steam, steam at atmospheric pressure, heat and moisture, or other accepted processes, it is allowable to accelerate strength gain and reduce time of curing. Accelerated curing should provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage. The curing process should produce concrete with durability at least equivalent to that required by the engineer or architect; strength tests should be performed to assure that curing is satisfactory.

The compressive strength of steam-cured concrete is not as high as that of similar concrete continuously cured under moist conditions at moderate temperatures. Also, the elastic modulus, E_c , of steam-cured specimens may vary from that of specimens moist cured at normal temperatures. When steam curing is used, it is advisable to base the concrete mixture proportions on steam-cured test cylinders. Accelerated curing procedures require careful attention to obtain uniform and satisfactory results. Preventing moisture loss during the curing is essential.

In addition to requiring a minimum curing temperature and time for normal- and high-early-strength concrete for judging the adequacy of field curing, at the test age for which the strength is specified (usually 28 days), field-cured cylinders should produce strength not less than 85% of that of the standard, laboratory-cured cylinders. For a reasonably valid comparison to be made, field-cured cylinders and companion laboratory-cured cylinders should come from the same sample. Field-cured cylinders should be cured under conditions identical to those of the structure—for example, if the structure is protected from the elements, the cylinder should be protected.

6.3.4.2 British Standard for Curing

Curing is the process of preventing the loss of moisture from concrete while maintaining a satisfactory temperature. The curing regimen should prevent the development of high-temperature gradients within the concrete. The rate of strength development at early ages of concrete made with supersulfated cement is significantly reduced at lower temperatures. Supersulfated cement concrete is seriously affected by inadequate curing and the surface has to be kept moist for at least 4 days. Curing and protection should start immediately after the compaction of the concrete to protect it from the following:

- premature drying, particularly by solar radiation and wind
- leaching out by rain and flowing water
- rapid cooling during the first few days after placement
- high internal thermal gradients
- low temperature or frost
- vibration and impact, which may disrupt the concrete and interfere with its bond to the reinforcement

When structures are of considerable bulk or length, the cement content of the concrete is high, or the surface finish is critical, special or accelerated curing methods are to be applied. The method of curing should be specified in detail.

BS 1881 states that surfaces should normally be cured for a period of not less than that given in table 6.2. Depending on the type of cement, the ambient conditions, and the temperature of the concrete, the appropriate period is taken from table 6.2 or calculated from the last column of that table. During this period, no part of the surface should fall below 5°C. The surface temperature is lowest as it rises and depends upon several factors, including the size and shape of the section, the cement class and cement content of the concrete, the insulation provided by the formwork or other covering, the temperature of the concrete at the time of placing, and the temperature and movement of the surrounding air. If not measured or calculated, the surface temperature should be assumed equal to the temperature of the surrounding air (see CIRIA report no. 43).

TABLE 6.2
Minimum Periods of Curing and Protection as in BS 8110

<u>Minimum period of curing, days</u>		Condition after casting	Type of cement
Any temperature between 10 and 25°C	5–10°C		
60/(t + 10)	4	Average	Portland cement and sulfate-resisting
80/(t + 10)	6	Poor	portland cement
80/(t + 10)	6	Average	All cement except the preceding and
140/(t + 10)	10	Poor	supersulfated cement
No special requirements		Good	All

Notes: Good = damp and protected (relative humidity greater than 80%; protected from sun and wind); average = intermediately between good and poor; poor = dry or unprotected (relative humidity less than 50%; not protected from sun and wind).

The most common methods of curing according to British specifications include:

- maintaining formwork in place
- covering the surface with an impermeable material such as polyethylene, which should be well sealed and fastened
- spraying the surface with an efficient curing membrane

6.4 PROTECTING SPECIAL STRUCTURES

The code and specification requirements in the design and execution of concrete structures to preserve the structure from corrosion are not sufficient in some structures exposed to very bad atmospheric factors that cause corrosion, such as in off-shore structures; special parts of the structure exposed to moving water (splash zone); or in parking garages or bridges, where salt is used to melt ice. There are some investments to establish modern commercial or residential buildings adjacent to sea-water that might not be exposed to seawater directly; those buildings are exposed to chlorides, as well as some marine areas with warm atmospheres—for example, in the Middle East, especially the Arabian Gulf area.

Large investment has been made in structures on islands or quasi-islands with high temperatures, as well in cities such as Singapore and Hong Kong. The corrosion has already started and the corrosion rate will be very high. Therefore, control of corrosion through the use of good concrete and by determining the exact content of the cement, as well as the specific w/c ratio, and caution in the process of curing and during construction are key factors for protecting reinforced concrete structures from corrosion. These are the first line of defense in attacking corrosion. Nevertheless, in some structures, these methods are considered inadequate, so other methods are used to protect structures from concrete corrosion; this will be clarified in the next chapter.

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7 Methods for Protecting Steel Reinforcements

7.1 INTRODUCTION

The minimum requirements in various codes are often insufficient to ensure long-term durability of reinforced concrete in severe exposures, such as those found in marine splash zones, bridges, and parking structures where de-icing salts are applied. In addition, some newer structures (such as commercial buildings and condominiums) built in marine areas, but not in splash zones, are experiencing corrosion problems due to airborne chlorides. Furthermore, marine structures in the warmer climates prevalent in the Middle East, Singapore, Hong Kong, South Florida, etc. are especially vulnerable due to the high temperatures, which increase not only the rate of chloride ingress, but also the corrosion rate once the process is initiated. In this chapter, a brief description of supplemental corrosion protection measures is given for structures especially at risk. Good-quality concrete as described in the next chapters is considered the primary protection method, but various combinations of this with supplements are necessary to reach the desired design life of the structure.

One of the most effective means to increase corrosion protection is to extend the time until chloride or a carbonation front reaches the steel reinforcement. The minimum code requirements, which allow the use of concrete with water-to-cement (w/c) ratio of less than 0.45 and concrete cover thickness of more than 38 mm, are totally inadequate for the structures and environmental conditions outlined previously if a design life of 40 or more years is specified. In many applications, designs complying with minimum code requirements would not provide even as little as 10 years of repair-free service.

Precaution is better than repair: Protecting the concrete structure is easier and less expensive than repairing it. Repairing and renovating some parts of reinforced concrete buildings (e.g., foundations) is very expensive. In reality, protection of the structure from corrosion is protection of the investment during the structure's lifetime. In the previous chapter, the process of corrosion of reinforced steel bars in concrete was discussed for different environmental conditions that cause corrosion; in addition, the different types of corrosion and their effects on the steel reinforcement were explained. In this chapter, our aim is to explain the different international methods available for protecting the reinforced concrete structure from corrosion.

Recently, more research and development has been conducted to provide economical and effective methods to protect steel reinforcement because this subject is very important to a construction investment considered to be worth a billion dollars worldwide. The first method of protection of steel reinforcement is to perform good quality control in design and construction, taking all precautions to avoid cases of

corrosion, as stated in codes and specifications in different countries. These specifications vary with the different weather conditions to which a structure is exposed, as well as with the function of the building. Offshore, underground, and surface structures are addressed by various important codes and specifications to protect the steel in their concrete members (see chapter 5).

The second line of defense in protecting the steel bars is by external methods. These methods include use of:

- galvanized steel bars, epoxy-coated steel bars, or stainless steel
- additives, such as cathode protection, that are added to concrete during the pouring process
- external membranes to prevent water permeability
- cathode protection

These methods have advantages and disadvantages. This chapter discusses them and provides ways of application for each type of protection.

7.2 CORROSION INHIBITORS

There are two types of corrosion inhibitor: anodic and cathodic. The anodic inhibitor depends on the protection of a passive protection layer on steel reinforcement. Cathodic protection is based on preventing the spread of oxygen in the concrete. Anodic inhibitors provide more effective protection than cathodic inhibitors and they are commonly used in practice. Next, we will explain each type of protection, along with its advantages and disadvantages.

7.2.1 ANODIC INHIBITORS

The most common anodic inhibitor is calcium nitrate, which is well known for its compatibility with the process of pouring concrete at the site and has no adverse impact on the properties of concrete, whether it is fresh or in a hardening state. Other types include sodium nitrate and potassium nitrate, which are highly efficient in the prevention of corrosion; however, they are not used because, in cases of existing aggregates with alkaline, they react with cement, causing many problems and extensive damage to the concrete's interaction with some cement. Broad calcium nitrate has been used widely since the mid-1970s (Bentur et al. 1998). Note that calcium nitrate accelerates the concrete setting time. Broomfield (1995) mentioned that some more retardant additives must be added to the concrete mix at the mixing plant; otherwise, there was no damage to the concrete.

The mechanism by which calcium nitrate inhibits corrosion is associated with the stabilization of the passivation film, which tends to be disrupted when chloride ions are present at the steel level. The destabilization of the passivation film by chlorides is largely due to interference with the process of converting the ferrous oxide to the more stable ferric oxide. The anodic materials are used with the concrete exposed to chlorides directly, as in contact with seawater. The corrosion inhibitor

TABLE 7.1
Amount of Calcium Nitrate Required to Protect Steel Reinforcements from Chloride Corrosion

Calcium nitrate amount (kg/m ³ and 30% solution)	Amount of chloride ions on the steel (kg/m ³)
10	3.6
15	5.9
20	7.7
25	8.9
30	9.5

reacts with chlorides and therefore increases the amount of chloride concentration necessary for corrosion, based on the tests described in table 7.1.

The amount of calcium nitrate to add is determined based on the amount of chloride exposed to concrete; this can be done in practice or through knowing the quantity of chloride from previous experience. The addition of corrosion inhibitors does not diminish the importance of attention to the quality control of concrete and maintaining the appropriate thickness of the concrete cover. In the case of high-strength, high-quality concrete, choosing and constructing the right concrete cover thickness and appropriate density of concrete according to specifications may obviate the need for a corrosion inhibitor for 20 years—although it is used when structures are exposed to chlorides directly (e.g., offshore structures).

7.2.2 CATHODIC INHIBITORS

Cathodic inhibitors are added to the concrete during mixing. A new type is painted on the concrete surface after it hardens. This type propagates to steel bars through the concrete porosity and provides isolation to reduce the quantity of oxygen—an important driver in the corrosion process—that spreads to concrete.

Many tests are performed for corrosion inhibition based on ASTM G109-92; their purpose is to define the effect of chemical additives on the corrosion of steel reinforcements embedded in concrete. From these tests, one finds that the corrosion inhibitor enhances the protection of the steel bars as well as controls concrete quality. However, cathodic inhibitors have less of an effect than anodic inhibitors. These tests have shown that, to obtain a higher efficiency from cathode protection for the reinforced steel bars, it is essential to add a very large quantity to the concrete mix. On the other hand, cathodic inhibitors such as amines, phosphates, and zinc slow the setting time considerably, especially when large quantities of cathodic inhibitors are used. Thus, when one decides to use a cathodic inhibitor, the concrete setting time must be taken into consideration.

From the previous discussion, one can conclude that an anodic inhibitor is more effective than a cathodic inhibitor. Therefore, in practice, the anodic inhibitor is generally used. If a cathodic inhibitor is used, the increase in concrete setting time should be kept in mind.

7.3 EPOXY COATING OF STEEL REINFORCEMENT

It is important to paint steel bars using certain types of epoxies able to protect steel from corrosion. This method yielded positive results, especially in steel exposed to seawater, in a study performed by the Federal Highways Association (FHWA) that has been evaluating the use of epoxy to coat steel reinforcements exposed to chloride attack. Also, some research, such as that done by Pike et al. (1972), Cairns (1992), and Satake et al. (1983), has demonstrated the importance of painting steel reinforcements. Epoxies have been used in painting reinforced steel for bridges and offshore structures since 1970.

Some shortcomings have been found using this method. Precautions must be taken during the manufacturing and operation of painting the steel, such as avoiding the absence of any friction between the bars, which would affect the result of the erosion of the coating layer due to friction. Also, it is difficult to use methods for measuring the corrosion rate such as polarization or half-cell, so it is not easy to predict the steel corrosion performance or measure the corrosion rate.

Painting steel-reinforced bars has been used extensively in the United States and Canada for 25 years. More than 100,000 buildings use coated bars, which is equal to 2 million tons of epoxy-coated bars. The coated steel bar must follow ASTM A 775M/77M-93, which sets allowable limits as the following:

- Coating thickness should be in the range of 130–300 μm .
- Bending of the coated bar around a standard mandrel should not lead to formation of cracks in the epoxy coating.
- The number of pinhole defects should not be more than six per meter.
- The damage area on the bar should not exceed 2%.

These deficiencies cited by the code are the result of operation, transportation, and storage. There are some precautions that must be taken in these phases to avoid cracks in the paint. Andrade et al. (1994), Gustafson and Neff (1994), and Cairns (1992) define ways of storing and steel reinforcement bending, carrying the steel, and pouring concrete.

Painting steel reinforcement bars will reduce the bond between the concrete and steel; therefore, it is necessary to increase the development length of steel bars to overcome this reduction in bond strength. According to American Concrete Institute (ACI) code (ACI Committee 318 1988), the increase of the development length is from about 20–50%. The American code stipulates that, in the case of painting, the development length of steel bars must be increased by 50% when the concrete cover is less than three times the steel bar diameter or the distance between the steel bars is less than six times the bar diameter; in other circumstances, the development length should be increased by 20%.

Egyptian code does not take painting steel into account. A study by El-Reedy et al. in 1995 found that the calculation of development length in the Egyptian code can be applied in the case of painting steel bars by epoxies without increasing the development length. However, the thickness of the paint coating must be governed by not more than the value cited by the American code (300 μm). Moreover, painting

TABLE 7.2
Corrosion Rate for Coated and Uncoated Steel Bars

Case	Corrosion rate (mm/year)	
	Tap water	NaCl 1% + Na ₂ SO ₄ 0.5%
Uncoated	0.0678	0.0980
Coated	0.0073	0.0130

the mild, low tensile steel at full bond strength is prohibited due to friction; when it is painted with coating, all the bond strength will be lost, so it is important to avoid coating the smooth bars. Care must also be taken not to increase the thickness of the paint coating to more than 300 μm. Some researchers have stated that when painting with a thickness of 350 μm was used for the main steel reinforcement in concrete slab, testing found too many cracks, which led to separation between steel bars and concrete.

Corrosion rates between steel bars without coating and other bars coated by epoxy were compared; both were exposed to tap water and then some samples were placed in water containing sodium chloride and sodium sulfate. Table 7.2 shows the rate of corrosion for the coated and uncoated steel bars. From this table, one can find that the corrosion rate is slower for coated than for uncoated steel bars. This method is cheap and is widely used by workers and contractors in North America and the Middle East.

Note that the coating of reinforcement steel by epoxy is not exempt from the use of concrete of high quality while maintaining a reasonably concrete cover. Some steel manufacturers can provide steel bars with the required coating. This is a good alternative to performing coating on-site because the thickness of the coating cannot be controlled as it needs some special tools to measure this thickness.

7.4 GALVANIZED STEEL BARS

Some research in the United States has recommended using galvanized steel in reinforced concrete structures. Moreover, an FHWA report recommended that the age of galvanized steel reach up to 15 years when high-quality concrete is used and under the influence of attacking chlorides (Andrade et al. 1994). Galvanized bar is used effectively in structures exposed to carbonation. Depletion of the galvanizing accelerates if galvanized bars are mixed with nongalvanized bars.

The process of galvanization is through the use of a layer of zinc. To illustrate this briefly, galvanization immerses the steel rod in a zinc solution at a temperature of 450°C and then puts it through a process of cooling. Hence, the zinc cover forms on the steel bar. This cover consists of four layers; the outer layer is pure zinc and the other layers are a mix between zinc and steel. Zinc is a metal in which corrosion will happen over time. The rate of corrosion under different weather factors can be calculated by the corrosion on the zinc layer and the time required for it to corrode. Note that the relationship between the layer thickness and lifetime is represented by a linear relation, as illustrated in figure 7.1. The galvanized coating must be tested

after bending the steel bars. When welding is used in manufacturing, the maximum zinc cover is around 200 μm , following ASTM A767/A767A M-90.

The stability of zinc essentially depends on the stability of the pH value in concrete. A pH equal to 13.3 is the value at which the passive protection layer forms, but when the value of pH increases, the zinc will melt until it vanishes completely—the situation illustrated in figure 7.2. Therefore, the process of galvanization is totally dependent on the proportion of pH in the concrete pores and relies mainly on the alkalinity of portland cement. Hence, the shortage of cover when galvanization pH is equal to 12.6 will be 2 μm ; in the case of pH equal to 13.2, shortage in the galvanized layer reaches 18 μm . This happens before the passive protection layer is present.

Laboratory tests have been conducted using different types of portland cement with different alkalinity. Assuming that corrosion is equal, when the thickness of the cover is equal to 60 μm , a lifetime of about 200 years can be expected when alkalinity is low and 11 years when it is high (Building Research Establishment 1969). Therefore, the thickness of the cover must be more than 20 μm in the American specifications (ASTM 767/A767 M-90). This specification is especially for galvanized steel bars, for which it identifies two types of galvanization: first class I and type II

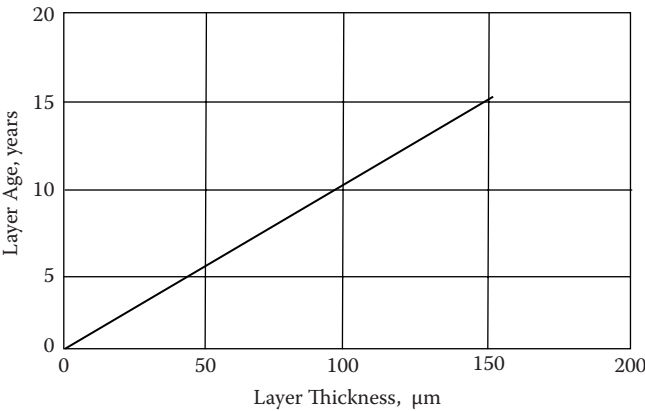


FIGURE 7.1 Relation between zinc layer thickness and expected lifetime.

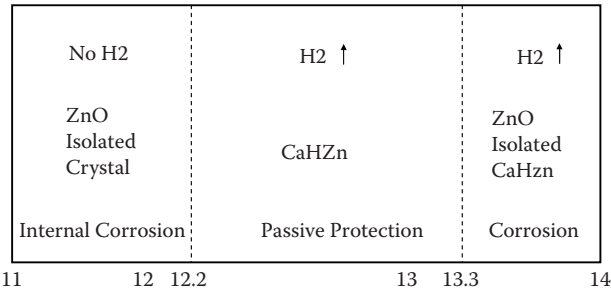


FIGURE 7.2 The galvanized steel behavior in solution with pH 11–14.

class, which have cover thicknesses of more than 1070 and 610 g/m², respectively—equivalent to 85 and 150 μm , respectively.

The United Kingdom's Building Research Establishment (1969) recommended maximum thickness for zinc cover of about 200 μm ; an increase in thickness of cover will reduce the bond between the steel and concrete. As stated before, when pH is between 8 and 12.6, zinc is more stable than for an increased pH value. The zinc gives high efficiency in the event that the structure undergoes significant carbonation, which reduces the pH, as we have discussed previously.

In the case of attacking the chlorides, galvanization will not prevent corrosion, but will significantly reduce the rate of corrosion. Galvanization can increase the value of chlorides, which can increase corrosion to 150–200%; this will increase the time to cause corrosion to four times longer (Yeomens 1994). The following are some of the important points that should be followed when using galvanization, according to the specifications of the Institute of Equatorial Concrete in 1994:

- Galvanization increases the protection of steel corrosion, but does not compensate for the use of concrete with an appropriate concrete cover.
- Galvanized steel should not be put with nongalvanized steel as this will quickly cause corrosion of the galvanic layer.
- It is necessary to examine the galvanic layer after bending the steel reinforcement and fabrication and to increase the bending diameter.
- Some precautions should be taken when using welding with galvanized steel.

7.5 STAINLESS STEEL

Some very special applications use reinforcing steel bars made from stainless steel material in order to avoid corrosion. This process is more expensive, so normal steel bars coated by a layer of stainless steel 1–2 mm thick are used. Here, recall the same precautions that have been mentioned for galvanized steel: Stainless steel or steel bars coated by stainless steel should not be put beside uncoated steel bars because this would lead to a fast process of corrosion.

In 1995 practice, some balcony designs used reinforcing stainless steel adjacent to normal steel and corrosion quickly followed (Miller 1994). But the high cost of the use of stainless steel makes its applications limited—about 8–10 times the cost of the normal steel commonly used. The cost of using stainless steel is about 15–50% higher than the cost of using epoxy-coated steel bars.

The specifications of steel coated by stainless steel are now being prepared and discussed by ASTM under the title, “Standard Specification for Deformed and Plain Clad Stainless Steel Carbon Steel Bars for Concrete Reinforcement,” which provides specifications for the installation of steel following specifications A 480/480 M for types 304, 316, or 316 L. Three levels of the yield stress are defined: 300, 420, and 520 MPa.

In the commercial market, reinforcing steel covered by stainless steel has yield strength of about 500 MPa, with maximum tensile stress about 700 MPa; the available diameters are 16, 19, 22, 25, and 32 mm. Some factories are now manufacturing 40-mm diameters. It is not preferable to use stainless steel when welding takes place,

but, if necessary, welding can be done by tungsten energy; the welding wire will be of the same materials as stainless steel.

7.6 FIBER REINFORCEMENT BARS

In the last decade, different theoretical and practical research has been conducted on the replacement of steel reinforcement bars by using fiber-reinforced polymer (FRP) because this material is not affected by corrosion and thus will be very economical during the course of a structure's lifetime. Figure 7.3 shows that the same shape of rolled steel section made from GFRP (glass-reinforced polymer) is more expensive than the traditional steel sections. On the other hand, there will be no cost for painting and maintenance work as they are not affected by corrosion under any environmental conditions surrounding the structure. Moreover, it has a light weight as the density of the fiber is 2.5 g/cm^3 ; the density of steel is about 7.8 gm/cm^3 , so its weight is three times less than the steel density.

Note that its weight is very low and its resistance is more than the resistance of steel, so it will be worth using to add more floors to an existing building. Now these sections are used to replace normal steel grating, handrails, and ladders in offshore structure platforms in the oil industry. These bars have been developed and manufactured from these materials to be alternatives to steel bars. Figure 7.4 shows that manufacture takes place in a certain way so that protrusions occur on the surface, which gives the ability to bond between them and the concrete. Currently, manufacture and use are on a small scale as they are under experiment in the market. Much research is being conducted to study the performance of these bars with time. As a result of limited production, the cost is very high; on the other hand, the maintenance is very low or can be ignored. Moreover, the value of the dead load to the structure will be reduced, which will reduce the total cost of the structure.

Some structures in Canada, such as marine structures in ports, or wharfs, have been established using bars of GFRP. These structures are composed of thick walls

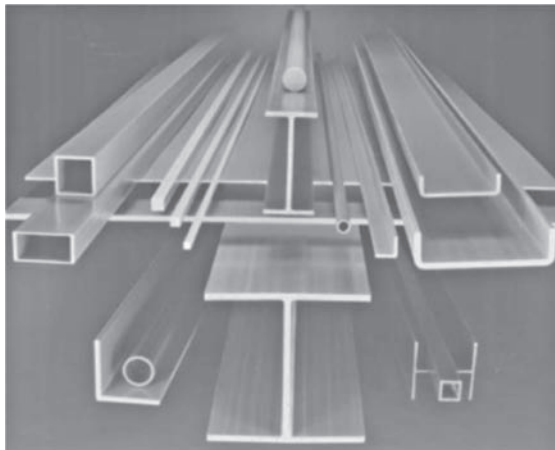


FIGURE 7.3 GFRP structural sections.



FIGURE 7.4 Shape of reinforcement bar from GFRP.

of precast concrete designed to have maximum strength of 450 kg/cm^2 . They are exposed to temperatures ranging from 35 to -35°C . Bridges in Quebec, Antherino, and Vancouver Island, Canada, for which salt has been used to dissolve ice, have been built using the same kind of reinforcement. Tests have been conducted on all these facilities through sampling; the reinforced concrete has been studied with an x-ray machine. These structures aged between 5 and 8 years. The research found that various factors, including wet and dry cycles, had no impact on the GFRP.

Microscopy examination found strong bonds between the bars manufactured from GFRP and concrete. The maximum strength was about 5975 kg/cm^2 and the maximum bond strength was 118 kg/cm^2 ; the modulus of elasticity was about five times less than the steel modulus of elasticity. To overcome such problems that might arise from the creep, the American specification ACI 440 has suggested that tensile stress must be not less than 20% the maximum tensile strength.

7.7 PROTECTING THE CONCRETE SURFACE

A concrete surface needs to be protected from permeability of water and propagation of chloride—for example, offshore structures exposed to sea waves, or bridge decks that use salt to melt ice; in addition, permeability needs to be prevented in order to protect concrete surfaces from carbonation and inhibit propagation to carbon dioxide. The other factors that increase the corrosion rate—humidity and oxygen—need to be prevented from spreading inside the concrete so that the corrosion rate will decrease. There are two popular methods of protection of concrete surfaces: spraying liquid materials or painting by brush and using sheets and membrane from rubber and plastic or textiles immersed in bitumen, which is usually used in small projects.

This isolation from water has the same problems for painting epoxy on reinforced steel as we discussed before, such as preventing any damage to this layer or erosion due to external mechanical factors such as tearing or existing heat sources and other factors. Thus, it is important to follow the required specifications during execution. Some specifications define the ways of execution in addition to the supplier's recommendations. There are different types of surface protection using liquid materials, but the function is the same: to prevent propagation of water inside concrete through concrete voids, as shown in figure 7.5.

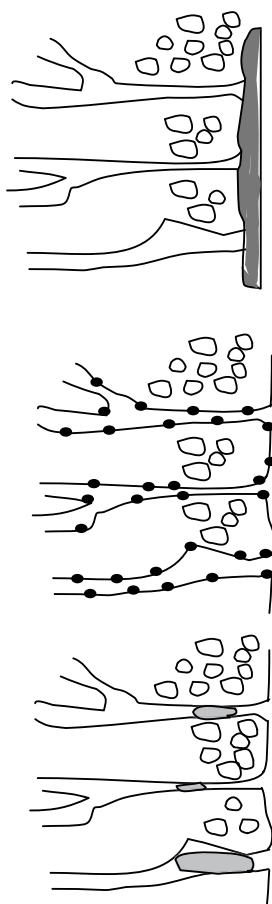


FIGURE 7.5 Sketch presenting different coatings.

7.7.1 SEALERS AND MEMBRANES

Sealers and membranes have been used traditionally for providing protection to concrete structures exposed to severe chemical attack. However, with growth in the frequency of durability problems generated by corrosion of steel in regular reinforced concrete structures, they are increasingly used as a means for mitigating this durability problem. Membranes and sealers can provide protection by (1) eliminating or slowing down the penetration of chlorides and carbonation, to keep the steel passivity; and (2) reducing moisture movement into the concrete to keep it dry and slow the spread of corrosion reactions. The sealers and membranes can be classified into several types and each type represents a family of materials with different chemical composition.

7.7.1.1 Coating and Sealing

These consist of a continuous film applied on the concrete surface with a thickness in the range of 100–300 μm . The film is composed of a binder and fillers for the

performance of the compounding of the fillers. Therefore, compositions having a similar binder may be quite different in their performance. The coating or sealers are obtained in a liquid form, which is brushed or sprayed on the concrete surface in an operation very much like painting.

The successful performance of coating or sealers depends not only on the quality of the materials used, but also on the application. The concrete surface should be clean and sound. Weak and cracked concrete should be removed, holes should be filled, and, if necessary, a leveling coat should be applied. If the membrane applied is of a polymeric composition, the surface should be dry. Silane treatments can be done in wet concrete. This is needed to facilitate better penetration of the liquid compound into the pores. Coating should usually be applied in two layers to obtain a continuous film without pinholes. The service life of the membranes or sealers ranges between 15 and 20 years. Thus, continuous maintenance and follow-up are necessary, including additional treatments.

7.7.1.2 Pore Lining

These treatments are based on processes by which the surface of the pores in the concrete is lined with materials that reduce the surface energy to make the concrete water repellent. Silicone compounds are most frequently used for this purpose. Silicone resin can be dissolved in an organic liquid that, after evaporation, deposits a resin film on the pore surface.

7.7.1.3 Pore Blocking

These treatments are based on materials that penetrate into the pores and then react with some of the concrete constituents. The resulting insoluble products are deposited in the pores to block them. The most common materials used for this purpose are liquid silicates and liquid silicofluorides. Experience has shown that using membrane to prevent water permeability has a lifetime of around 15 years; after that, it must be replaced. However, there are some problems (e.g., bathrooms) when this proactive maintenance is performed.

The proactive approach is very expensive because it needs to remove all the tiles and plumbing accessories and start from scratch; therefore, in residential buildings, the maintenance will be corrective. In any mature reinforced concrete structure, most of the problems due to isolation will be found because the main steel of the slab is lower and the signs of corrosion will be parallel cracks to the steel bars in the main steel bars. Such signs will be seen on the lower floor, so, for example, a user of a bathroom cannot observe this. Therefore, the repair will be done until the concrete cover falls and serious corrosion problems occur in the steel bars. When the membrane is used, one must take care in execution as any tear in the membrane or bad execution that does not follow the specifications and supplier recommendations will lead to losing money. Any defect will cause permeability of water and start corrosion.

There are two well-known examples of application of isolation for foundations: swimming pools and tanks. Here, methods isolate the surface that is exposed to water or isolate the surface that is not exposed directly to water. These types of isolation must be executed by a professional contractor experienced in this type of work

because using the chemicals and dealing with the membrane need competent workers available only from a trustworthy, specialized contractor. Recently, some research has found that problems occur in using isolation by membrane in cases of cathodic protection because some gases accumulate in the anode and need to escape; however, the membrane will prevent this and thus prevent completing the electrical circuit.

7.7.2 CATHODIC PROTECTION BY SURFACE PAINTING

The corrosion of steel in concrete occurs as a result of attacking chlorides or carbonization of a concrete surface and incursion into the concrete until it reaches steel, which reduces concrete alkalinity. The presence of moisture and oxygen bring about practical corrosion and will continue until a complete deterioration of concrete takes place, as has been clarified in chapter 2. Some materials are used to paint the surface of concrete to a saturation point. Then this material will spread through the concrete to reach steel at speeds of 2.5–20 mm per day up to the surface of steel by capillary rise, such as water movement; through penetrating concrete with water in cases of chloride attack; or through propagation by gas such as carbon dioxide when exposed to the surface carbonation.

Therefore, when these materials reach the surface of steel, an isolation layer around the steel bar's surface will reduce the oxygen in the cathode area to the surface on the cathode and reduce the melting of steel in the water in the anode area, thereby delaying the process of corrosion and reducing its rate. Figure 7.6 presents

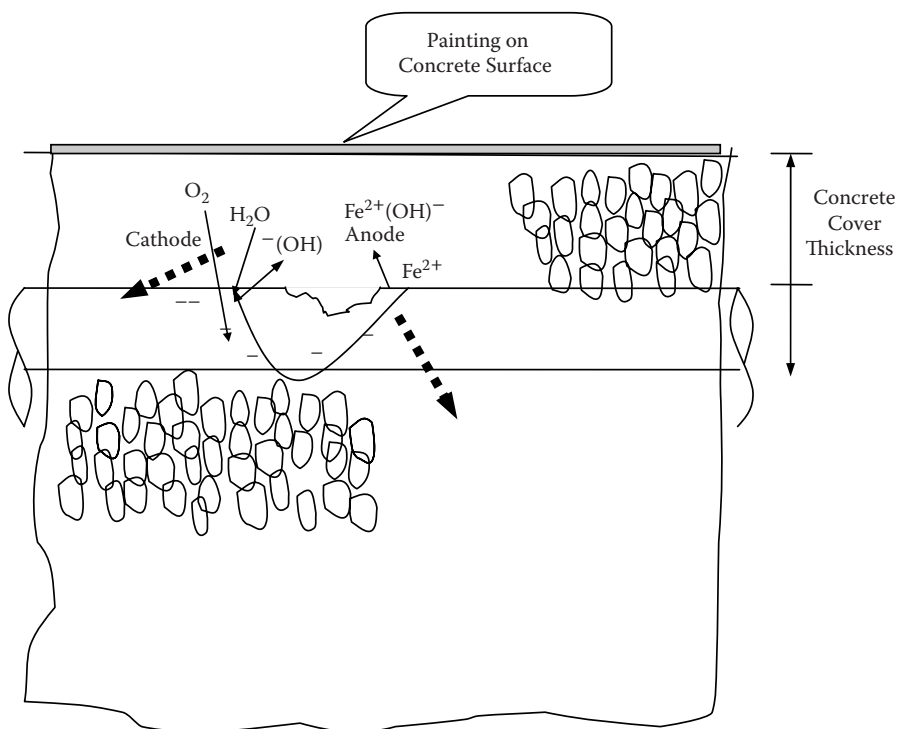


FIGURE 7.6 Painting a concrete surface to provide cathodic protection.

the influence of this cathodic protective coating on the surface in protecting the steel bars. These new advanced materials are used for new construction or for existing structures in which corrosion has started in steel bars. They are also used when corrosion is clearly present and repair work is necessary that requires complete repair to the damaged concrete surface, followed by painting the surface.

7.8 CATHODIC PROTECTION SYSTEM

This method is the most expensive method of protection; it is used usually in protecting pipelines in the petroleum industry. It is intended for use in reinforced concrete structures and also for special structures due to its higher cost and need for special studies, design, execution, and monitoring. If applied properly, cathodic protection can prevent corrosion of steel in concrete and stop corrosion that is already in progress. It accomplishes this by making the steel bar a cathode by use of an external anode. Electrons are supplied to the reinforcing bar from the anode, through the ionically conductive concrete. The current supplied should be sufficiently high so that all the local cells are inhibited and the entire steel surface becomes anodic. The external current can be supplied by connecting the steel to a metal that is higher in the electrochemical series (e.g., zinc). This serves as the anode relative to the cathodic steel. In this method, the anode gradually dissolves as it oxidizes and supplies electrons to the cathodic steel. This type of cathodic protection is called a “sacrificial” anode.

An alternative method for cathodic protection is based on supplying electrons to the reinforcing steel from an external electrical power source. The electrical power is fed into an inert material, which serves as the anode and is placed on the concrete surface. This method is referred to as “impressed current anodic protection.” The anode is frequently called a “fixed” anode.

7.8.1 CATHODIC PROTECTION

The principle of the use of a sacrificial anode was put forth in 1824 by Sir Humphrey Davy. Afterward, the discoveries were used for boats completely submerged under water to protect their metal parts from corrosion. At the beginning of the twentieth century, the technology was used for underground pipelines, but when it was discovered that the soil's resistance to electricity use was very high, cathodic protection with the current and constant direct current. Cathodic protection is used in the construction of special modern structures having more importance. The first practical application in the area of reinforced concrete was a bridge on a highway in the mountainous area north of Italy.

The use of cathode protection became more widespread with the development of research and technology. As Broomfield (1995) stated after his survey of consultants working in the area of cathode protection of concrete structures in England, protection of cathodic disabilities has been established for about 64,000 m² in about 24 structures in the United Kingdom and the Middle East. The largest manufacturer of anodes has supplied anodes that cover about 400,000 m² around the world and include a garage for 60 cars and 400 different forms of bridges and tunnels.

Cathodic protection is used in a certain type of construction as a result of high cost as well as the specific nature of the construction and the need for a system of monitoring. It is used most often in structures exposed to chlorides that exist inside the concrete mix, or penetration of chlorides within the concrete resulting from surrounding environmental conditions. Chloride influence in the presence of concrete has a special nature in that it starts when corrosion results from chlorides and the part of the concrete contaminated by chlorides must be completely removed.

Sometimes, the work of repair is very difficult. The case of chlorides in the concrete mix is considered impossible. Experience has found that the limited availability of electrical protection is more effective in stopping the process of corrosion, as mentioned in some previous studies, than the traditional way in the case of concrete pollution by chloride. The FHWA in the United States has said that the method of repair proven to be the only way to stop corrosion in concrete bridges with salts is use of electrical current, irrespective of the chloride content in concrete.

The use of cathode protection specifically prevents the corrosion of steel reinforcement in concrete or stops the corrosion process when it has already begun; this method relies on making steel with a continuous cathode through the use of an external anode (see fig. 7.7). As seen from this figure, when cathodic protection is not used, a cathode with a negative electron will form on the surface of a steel part and the other part will work as an anode; from this, the corrosion process forms, as explained in chapter 4. As shown in figure 7.7, the electron will generate on the steel reinforcement surface when an anode is placed on the concrete surface. In this case, the concrete will be electrically conductive between it and the steel reinforcement, so the cathodic protection will be formed on the steel reinforcement. As shown in the figure, the positive anode will move to the anode. This method of cathodic protection is called sacrificial protection and the anode in this case is called a sacrificial anode.

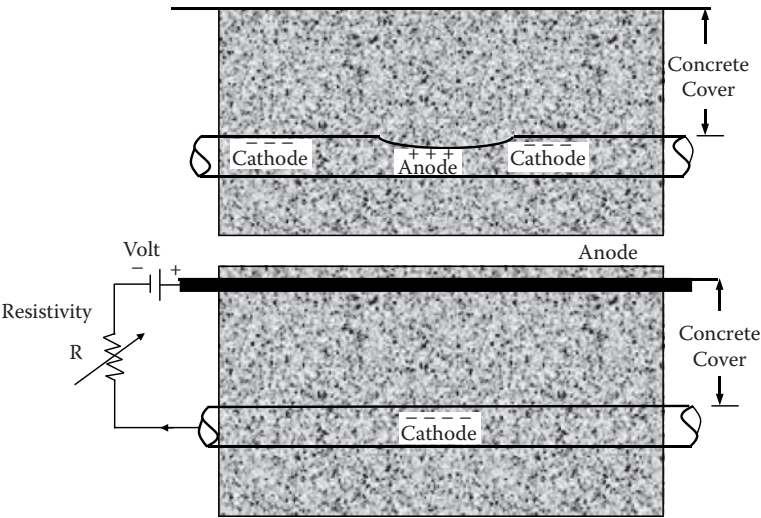


FIGURE 7.7 Cathodic protection method.

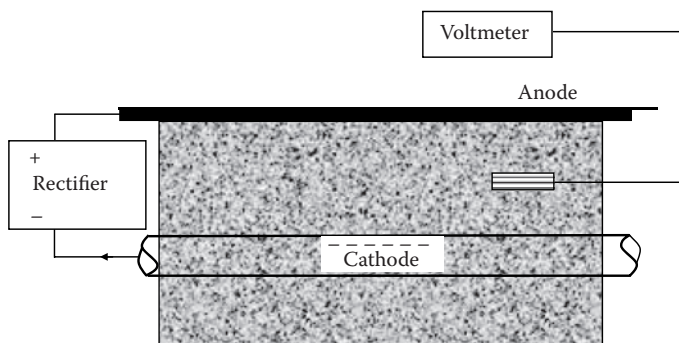


FIGURE 7.8 Cathodic protection method.

There are two main methods for cathodic protection. The first depends on using a sacrificial anode and is called sacrificial protection. In this case, the anode will be from zinc and the corrosion will happen to zinc instead of to the steel reinforcement. The oxidation process will cause the zinc to move to the steel reinforcement and the negative electron will form upon it; this is the required cathodic protection.

The second method of cathodic protection will generate electrons on the steel reinforcement in concrete by using outsources of electricity. This source will be conducted with the anode and will be put inside the concrete and called a fixed anode. Sacrificial protection is used in submerged structures; the concrete is immersed in water so that there will be little electron movement and the potential voltage between the two materials will be small. This will maintain the cathodic protection for a long time.

Figure 7.8 illustrates the second method of protection—putting an anode in concrete and using an external source of electricity. An example of a fixed anode is wire mesh put in concrete at the concrete surface; it works as anodes; the conductivity among anodes, steel reinforcement, and batteries by using cables is shown in figure 7.8. The source of energy will be through normally used batteries. A RILEM report put some consideration into more valuable cathodic protection:

- The electrical conductivity to the steel reinforcement must be continuous.
- The concrete between steel reinforcement and anode must be able to conduct electricity.
- Alkaline aggregates must be avoided.

7.8.2 CATHODIC PROTECTION COMPONENTS AND DESIGN CONSIDERATION

The cathode protection system consists of impressed current, anode, and the electric conductive element, which is the concrete in this case. It depends on the concrete relative humidity as it has a great effect on the electrical cables and the negative pole system on the steel reinforcement. The last component of this system is the wiring that will conduct between the anode and the source of direct current. But we must not forget the last important element, which is the control and measurement system. The most important and expensive element in the cathodic protection system is the anode, as it should be able to resist the chemical, mechanical, and different environmental

conditions that affect it over the structure's lifetime. In general, its lifetime is preferable to its coating layer life. The anode surface must be large and have a little density to the electric current so as to allow any deterioration to the anode.

From a previous discussion, it is clear that cathodic protection of a concrete structure is different from any other applications because, in some parts, concrete has pores that avoid containing water and other parts may be dry or have water on the surface only; thus, marine structures and structures buried in soil are different. The main fact is that the concrete structure cannot contain 100% water inside its pores. This problem is clear if the concrete around the anode is completely dry; the potential volts need to be increased by using 10–15 V instead of 1–5 V. On the other hand, the dry condition around the steel bars prevents causes of corrosion. Practically speaking, this cannot happen and only can be seen in summer without high relative humidity.

7.8.2.1 Source of Impressed Current

Most design methods take a current of about 10–20 mm ampere on the surface meter for steel reinforcement and take into consideration the lower layer of steel reinforcement. Potential voltage of around 12–24 V is always used to ensure that humans or animals experience little electric shock. The source of electric current is chosen in the design stage; the choice of electric source will be based on the fact that it must be enough to stop the corrosion process. A higher estimate for electric current is often assumed and 50% is added as a safety factor. This increase in current will produce little heat that will lose some part of the electric current.

7.8.2.2 Anode System

The most important element in the cathodic protection system is the anode. As the anode is the most expensive element of the system, experience in choosing and installing it is also needed. The choice of the anode system depends on the type of the structure, its shapes, and other requirements. There are two main types of anodes: The first type is used on bridge decks and fixes the deck from the top surface; it needs special properties to accommodate moving vehicles and will be covered by a layer of asphalt. The second type is used in vertical buildings; these anodes do not need high resistance to abrasion, as the first type does.

Anodes for bridge decks. From an execution point of view, there are two types: The first buries the anode in the concrete top layer, as shown in figure 7.9. The main problem with this type is that it increases the covering layer that will coat the anode, which consequently will increase the dead load on the structure. Therefore, this is not an economic solution. The second executable type is by making holes on the concrete surface and installing the anode inside. This method can overcome the increase of dead load on the structure more easily than the first type. On the other hand, it will be difficult to cut parts of the concrete as the anodes should be adjacent to each other by about 300 mm to maintain the suitable distribution of electricity. The first type can use carbon anodes or silicon iron with 300-mm diameter, 10-mm-thick discs buried inside asphalt coke; the upper layer is an asphalt conductive layer as a secondary anode. The second type, as shown in figure 7.10, makes 300-mm-diameter holes on

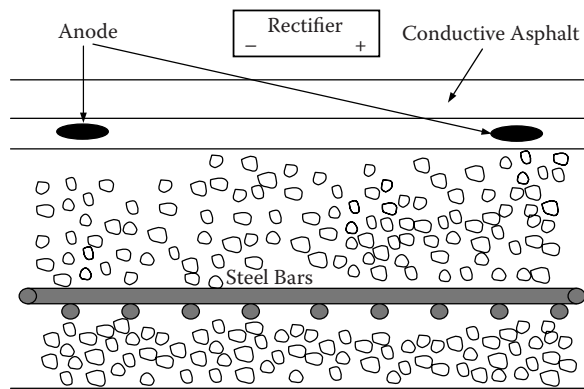


FIGURE 7.9 Sketch for using conductive asphalt.

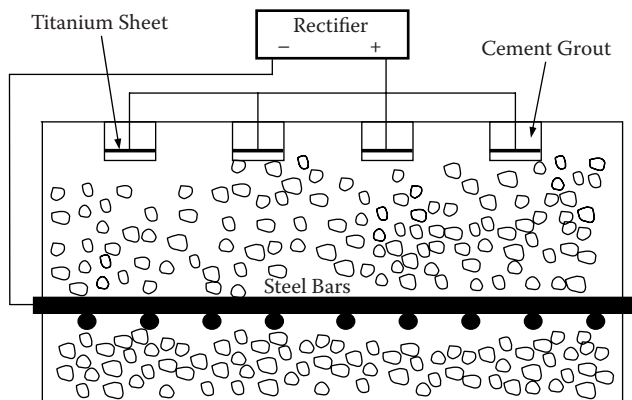


FIGURE 7.10 Sketch presents anode holes at the surface.

the upper concrete surface; the main target is to reduce the dead load on the bridge, consequently reducing the overall construction cost and reducing the increase of the bridge surface leveling.

Vertical surface anodes. In this type of concrete structure member, titanium mesh is always used and often concrete is applied by using shotcrete on the surface of the mesh to cover the anode. Strong precautions are needed in using shotcrete as it needs more competent workers, supervision, and contractors. It also needs special curing to guarantee good-quality concrete and good adhesion between it and the old concrete surface. Some precautions must be considered during construction to ensure that the layer saturated 100% will bond with old concrete and the electric conductivity will be through pieces of titanium sheet that will weld on the mesh, as shown in figure 7.11.

In a way, we may use the rod of titanium platinum, placing inside the concrete member of a subject inside the coke so as to reduce the impact of acid generated on the anode. One of the most important precautions during implementation is to make sure that there is no possibility of causing a short circuit between anode and steel



FIGURE 7.11 Fixing titanium on a concrete column.

reinforcement. To prevent that, through the use of appropriate cover, after making a hole in the concrete, the cover will give warning when it comes into contact with steel bars, as in figure 7.12. The anodes should be distributed by design system in order to ensure the protection of steel reinforcement in the structure. This method is used also in horizontal surfaces.

7.8.2.3 Conductive Layer

The anode is a main pole of inert metal; the secondary poles consist of layers from mortar, asphalt, and coating. This layer conducts the electric current by the carbon particles. Figure 7.13 shows the conductive layers in the case of vertical reinforced concrete structures like columns and walls. These layers do not have the capability to be durable with time, as the titanium mesh does, but are considered the most inexpensive and easiest way from a construction and maintenance point of view. They can be painted by a reasonable painting match with the architectural and decoration designs for the building, ceiling, and vertical and horizontal surfaces. The main disadvantage is that they cannot be used in a surface exposed to abrasion as capacity is low and their lifetime about 5–10 years.

7.8.2.4 Precautions in Designing the Anode

The most important and most expensive part of a system of cathodic protection is the anode. Therefore, some precautions and specifications are required, including selection of a type appropriate to the structure; every type has advantages and disadvantages, so it needs to be studied based on the application. As a result of the chemical reactions, the alkalinity of the concrete on the surface decreases, increasing the acidity. The material using the anode may have a high resistance to acid, but

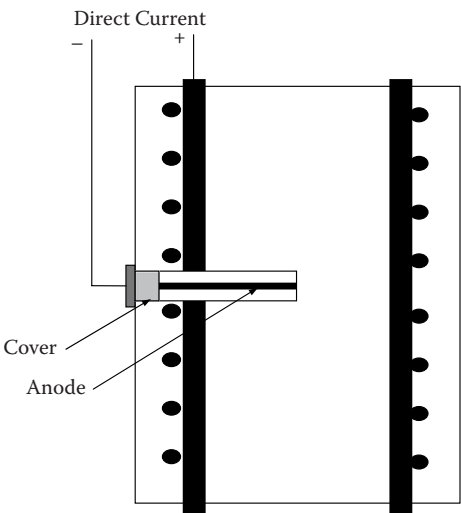


FIGURE 7.12 Sketch represents a buried anode.

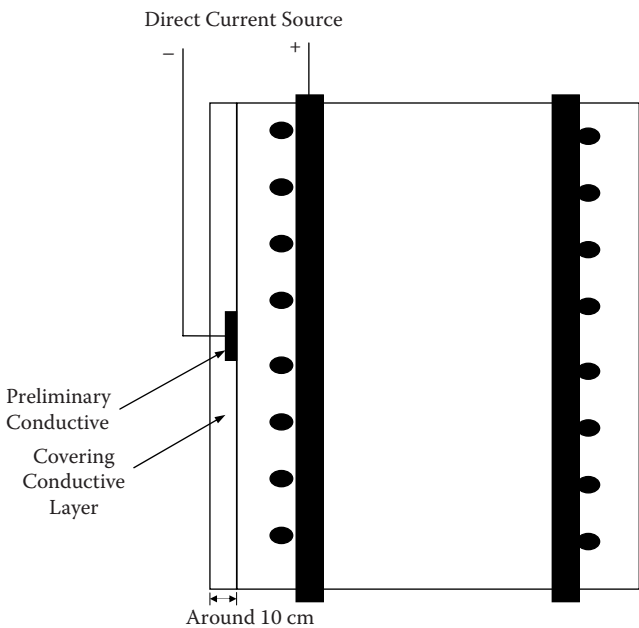


FIGURE 7.13 Present anode conductive layer.

the surrounding concrete may not have this capability, so it must be considered as the amount of accumulated acid proportional to the density of the electric current. For example, current density of 1.1 ampere/m² and surface area equal to steel and concrete results of reactions will cause corrosion of about 0.3–0.4 mm from concrete in general. In the case of using shotcrete with cement to aggregate, the ratio is 1:5; in the case of increasing the current density, the deterioration rate will increase in

addition to that for existing chlorides, and increasing it has an effect on the increasing acid rate. Therefore, increasing the area of anode to concrete surface area will be better from anode consumption and the acid formation.

7.8.2.5 Follow-Up Precaution

Note that the cathode protection needs to be followed up periodically to ensure that the cathode protection system continues regularly without any obstacles, as in figure 7.14. The cell pole near the steel reinforcement (see figure) can be connected easily by a microprocessor via cable modem, which can measure and transfer data to the computer directly. Consequently, this system can work without periodic visits to the site. The computer, which has special software and hardware, can monitor more than one point in the structure. This computer can follow up on the efficiency of the cathodic protection. Advanced systems can be used to control the source of the electric current by changing the current density and potential from a long distance away by using the microprocessor.

7.8.3 COMPARISON BETWEEN CATHODIC PROTECTION AND OTHER TYPES OF PROTECTION

Cathode protection needs to be measured with each period of time or by measuring away from the structure, as we have said before, using the computer to ensure the safety of the design and the proper functioning of the cathode protection. Therefore, cathode protection is more costly than other types of steel reinforcement protection and is often used in important structures with a special nature. Hence, it has been used in bridges and tunnels since the late 1980s and early 1990s, including an underground garage at the World Trade Center in New York City, as well as structures in the Arab Emirates. Kendell and Dailey (1999) reviewed the implementation

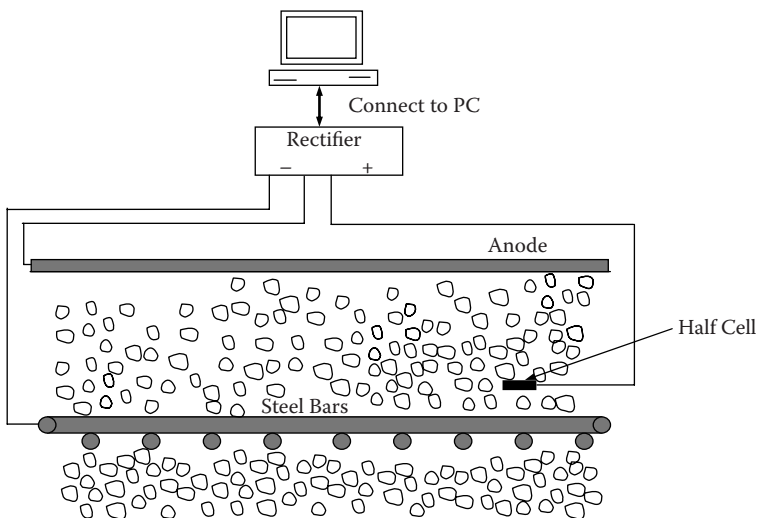


FIGURE 7.14 Cathodic protection monitoring.

method of cathodic protection during the implementation of the civil works, noting that it does not need excess labor for implementation as anodes are put with reinforcing steel (see fig. 7.15a). Figure 7.15b shows the development of titanium mesh with reinforcing steel and how to stabilize the titanium mesh through a link from the plastic. Figure 7.16 shows titanium bars; this method does not need extra workers as the workers that place the steel reinforcement will place the titanium, also.

7.8.4 CATHODIC PROTECTION FOR PRESTRESSED CONCRETE

Most specifications state that cathode protection must not be used when prestressed concrete is used. As was explained before, the movement of electrons forms hydrogen ions on the surface of a steel reinforcement. The presence of hydrogen affects composition of iron atoms for the high-strength steel that is usually used in the process of pretension of the steel in cases of prestressed concrete. Cathodic protection can be used, taking into account that hydrogen will not affect the nature of iron and the quantity of hydrogen resulting from the cathode protection process does not pose a threat to the prestressed steel stress. Some prestressed structures do not use cathode protection of underground structures. Some cases in Florida have used



FIGURE 7.15a Fixing an anode on a bridge deck.



FIGURE 7.15b Fixing titanium mesh by plastic binder.

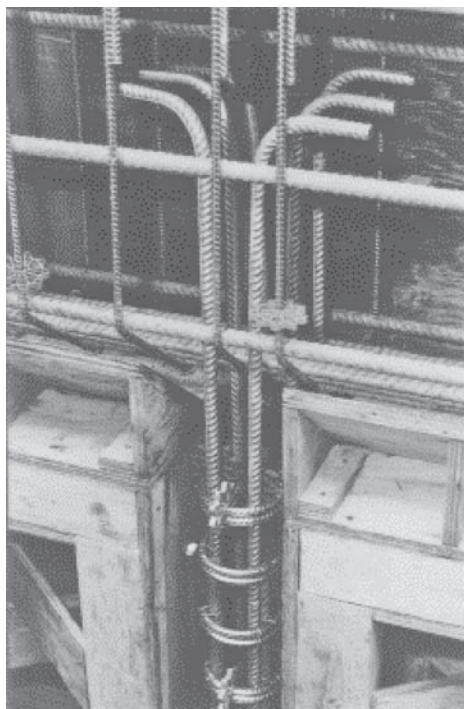


FIGURE 7.16 The installation of titanium bars with steel bars.

sacrificial anodes in bridge pilings by using depleted anodes (Kessler, Powers, and Lasa 1995).

7.8.5 BOND STRENGTH IN CATHODIC PROTECTION

The impact of electric current on bond strength between steel reinforcement and concrete has been considered one of the most important subjects studied by researchers for several years. From a practical point of view, there are no notes for any impact of cathodic protection on bond strength. Some structures that have been studied are about 20 years old and there has been little impact on the bond strength, as most of it is carried out by the ribs that exist in the deformed steel reinforcement.

Some practical experiences have proved that bond strength increases when rust is present on the steel surface. When cathode protection is used, there is no need to use another protection method, such as membrane, which prevents water permeability, or some additions or painting the steel bars. This reduces the total cost of the project and, for that, in deteriorating concrete structure or some of its members, it was used at a high cost. Therefore, the cost of repairing the concrete member may be higher than the cost of constructing it.

In some structures, due to the special nature of construction, it is difficult to work the necessary repairs, so the cathode protection is the best and only solution. Table 7.3 contains a summary of the methods of protection prepared by Kendell and

TABLE 7.3
Advantages and Disadvantages of Different Types of Protection

Method	Advantage	Disadvantage	Compatible with cathodic protection?
Increase concrete cover	No cost increase Obtain from good design	Has limits Increasing cover will increase the shrinkage crack	Yes
Not permeable concrete	Depend on design	Need additives with very good curing	Yes, but silica fume must be <10% cement weight
Penetrating sealer	Relatively no cost	Complicated practically	Yes
Membrane prevents water	Very well-known technology	Can be damage Difficult to fix without fault	Yes, but cathodic protection under the membrane
Epoxy-coated steel bars	Well-known technology Little maintenance	Problem in quality control under certain weather conditions	Yes, with continuous electric current
Galvanized steel	Easy use on-site	Quickly damaged by contact with nongalvanized steel	Yes; need higher level of protection
Stainless steel	Excellent for corrosion prevention	High cost	Yes
Corrosion inhibitor	Calcium nitrate effect for little chloride concentration	Must know the chloride amount Effect over a long time not known	Yes
Cathodic protection	Well-known technology Long life by titanium anode	Regular inspection of rectifier	

Daily in 1999; it gives the advantages and disadvantages of each method and the appropriateness of use with the cathode protection at the same time.

7.9 CONCRETE WITH SILICA FUME

Concrete is generally classified as normal-strength concrete (NSC) and high-strength concrete (HSC) but there is no clear-cut boundary for the two classifications. International codes generally define HSC as concrete with strength equal to or more than 50 N/mm². Recently, a new term, “high-performance concrete” (HPC), has been used for concrete mixtures that provide concrete with high workability, high strength, high modulus of elasticity, high density, low permeability, and resistance to chemical attack.

There is a little controversy between the terms “high-strength” and “high-performance.” High-performance concrete is also a high-strength concrete, but it has a few more attributes specifically designed, as mentioned earlier. But, in terms of concrete durability, HPC is very important as its permeability is very low. Therefore, the spread of carbon dioxide, oxygen, or chloride will be very small, which will enhance the durability of the concrete from corrosion.

The main process to obtain HPC is to reduce w/c ratio to less than 0.3, which will greatly improve quality expected from the HPC. The main player to obtain high workability and high strength is the use of silica fumes, which become a necessary ingredient for strength above 80 N/mm². The ACI defines silica fume as a “very fine noncrystalline silica produced in an electric furnace.” Also referred to as microsilica or condensed silica fume, this product results from reduction of high-purity quartz with coal in an electric arc furnace during the manufacture of silicon or ferrosilicon alloy. Silica fume rises as an oxide vapor. Condensed silica fume is essentially silicon dioxide (more than 90%) in noncrystalline form. It is considered an airborne material with a spherical shape. It is extremely fine, with particle sizes less than 1 μm and with an average diameter of about 0.1 μm , so it is about 100 times smaller than average cement particles. Silica fume has a specific surface area of about 20,000 m²/kg; cement has a surface area from 230–300 m²/kg. Due to its very small size, silica fume can close any pore void in concrete, so it enhances the properties of concrete by high strength and low permeability—the two main keys to having durable materials. Note that silica fume is more expensive than normal concrete strength due to use of silica fume and superplasticizer.

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8 Repair of Reinforced Concrete Structures

8.1 INTRODUCTION

In most cases, repairing and retrofitting a concrete structure is much more complicated than building a new structure because new constructions do not need competent engineers with a lot of experience, but rather can be handled by junior engineers with a reasonable amount of experience (2 or 3 years). On the other hand, repair of concrete structures is more challenging and needs competent engineers and consultants as well. The building already exists and it is necessary to define new solutions to the problem that must be matched with the nature of the building and owner requirements, and satisfy safety and economic requirements at the same time. The repair process for reinforced concrete structures is very important and dangerous; extensive care must be taken in choosing the suitable repair methods and tools.

The reasons for structure deterioration can be summarized as follows:

- Code and specification precautions were not considered in the design phase.
- Code and specification precautions were not considered in the execution phase.
- A suitable protection method was not selected in the design phase.
- A suitable protection method was selected, but the execution was bad.
- No maintenance monitoring was performed.

In all these cases, the result will be the same: structure deterioration, which will be obvious by seeing the cracks in and concrete cover falling from the different reinforced concrete elements in the structure. When cracks appear and concrete cover falls, the situation becomes very bad. Shortages occur in a cross-sectional area of the concrete member, which requires speeding up the work of repair to avoid a worsening structure condition, most likely leading to collapse with time. The first and most important step for any repair is to determine accurately what will be repaired; this depends entirely on assessment of the structure and answers to these questions:

- What is the reason for corrosion?
- Are cracks and structure deterioration on the increase?
- What is the expectation of the extent of deterioration of concrete and extension along the corrosion in the steel?
- What is the impact of current and expected future deterioration on the safety of structure?

The answers to these questions will need to use structural assessment methods and different measurement methods clarified in chapter 4. These will help in deciding the cause of corrosion and present and future deterioration of the structure.

It is worth noting that the process of repair and restoration of concrete structures from the assessment phase to the execution phase needs a high level of experience. When conducted by inexperienced engineers, even the simple evaluation stage may lead to the wrong choice of repair method, which is a big problem. When structures are assessed, several strategies for repair contained in a special report by the RILEM Committee (1994) clarify the various repair strategies used in most cases:

- re-establishment of the deteriorated member
- comprehensive repair of the concrete member to regain its ability to withstand full loads
- repair of a particular portion of the concrete member, followed up continuously over periods of time
- strengthening structures by an alternative system to bear part of the loads

In most cases, the strategy of repair is either a comprehensive or a partial repair of the concrete member. These strategies are common in the rehabilitation of concrete and they depend on the structural system, external environmental factors, and the degree of structural degradation.

8.2 MAIN STEPS TO EXECUTING REPAIR

There are several regular steps in the repair of all structures exposed to corrosion. The very critical first step is to strengthen the structure by performing structural analysis and designing a suitable location for the temporary support. The second step is to remove the cracked and delaminated concrete. It is important to clean the concrete surface and also the steel bars by removing rust. After rust is removed by brush or sand blasting, the steel bars should be painted with epoxy coating or replaced; then new concrete can be poured. The final step is to paint the concrete member as external protection. This is a brief description of the repair process. These steps will be explained in detail in the following sections.

8.2.1 STRUCTURE STRENGTHENING

One of the most dangerous and important first steps necessary for the repair is selecting the temporary support, which depends on the following:

- evaluating the state of the whole structure
- determining how to transfer loads in the building and its distribution
- determining the volume of repair that will be done
- determining the type of concrete member that will be repaired

As mentioned earlier, the repair process must be carried out by a structural engineer with a high degree of experience who has the capability to perform the structure

analysis and powerful knowledge of the load distribution in the structure, according to the kind of repair. The structural engineer has a responsibility to choose the right way to optimize the process of crushing and to determine the ability of the structure members to carry the loads that will be transferred to them. Therefore, the responsible engineer should design the temporary support based on data collected and the previous analysis and should be cautious in the phase of execution of temporary supports.

Choosing how to remove the defective parts will be based on the nature of the concrete member in the building as a whole; any member of breaking concrete has a detrimental impact on neighboring members because the process of breaking will produce a high level of vibration. The temporary members must be strong and designed to withstand loads and must be transported easily and safely to the defective area. The entire structure depends on the design and execution of the temporary supports and their ability to bear loads safely.

8.2.2 REMOVING CONCRETE CRACKS

There are several ways to remove the part of the concrete that has cracks on its surface and shows the effects of steel corrosion. These methods of removing the delaminated concrete depend on the ability of the contractor, specifications, the cost of breaking, and the whole state of the structure. The selection of the breaker methods is based on the cause of corrosion; if it is due to carbonation or chlorides, then one must also consider whether cathodic protection will be performed in the future. In this situation, the breaking work would take place on the falling concrete cover; it would be cleaned and all the delaminated concrete and cracked concrete parts removed. Then, high-strength, nonshrinking mortar would be poured.

If the corrosion in steel reinforcement is a result of chloride propagation into concrete, most specifications recommend removing about 25 mm behind the steel and making sure that the concrete on the steel has no traces of chlorides after the repair process. The difference between good and bad repair procedures is obvious in figure 8.1.

The difference in the procedure of breaking the delaminated concrete is due to the difference in the causes of corrosion. Therefore, a careful study to assess the state of the structure and the causes of corrosion is very important to get high quality after the repair process. The evaluation process is the same as illness diagnosis. If there

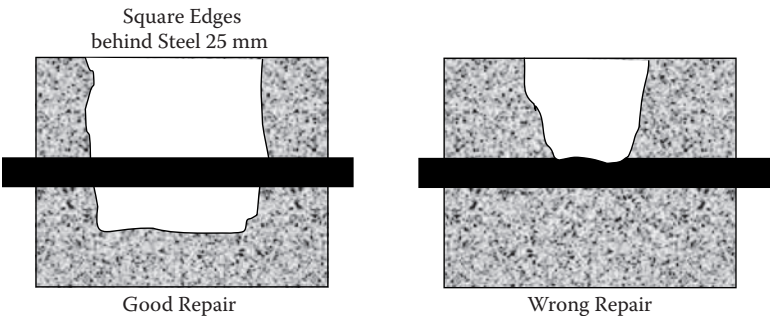


FIGURE 8.1 Differences between good and bad repairs.

are any mistakes in the diagnosis, the repair process will be useless, as well as a waste of time and money. It is necessary to define the work procedure and quantity of concrete that will be removed. This step is considered one of the fundamental factors for designing and installing wooden pillars of a building to be used during repair. Therefore, the work plan must be clear and accurate for all the engineers, foremen, and workers who participate in the repair process.

After competent staff are secured and the building assessment and design of supports and ties are completed, some information may not be available—for example, the construction procedure for this building or the workshop drawings or specifications followed when the building was constructed. Therefore, the risks are still high. The only factor that can help in reducing this risk in spite of the steel corrosion is the increase in concrete strength with time. However, this compensation occurs within a limit because the steel is carrying most of the stresses and the risk will be very high in cases of spalling the concrete cover due to reduction in concrete cross-sectional dimensions.

It is necessary and important to remove concrete for a distance greater than the volume required for removal of defective concrete so that proper steel can be reached. This will be important later in the repair process. Several methods are commonly used for breaking and removing the defective concrete and these are explained next.

8.2.2.1 Manual Method

One of the simplest and easiest methods is to use a hammer and chisel to remove defective concrete. This is considered one of the most inexpensive ways, but it is too slow compared with mechanical methods. However, mechanical methods produce high noise and vibration, have special requirements, and need trained labor. Using the manual method makes it difficult to spare concrete behind the steel. The method is used in the case of small spaces and is the preferred use in the event of corrosion due to carbonation and attacking chlorides from outside, when it is not necessary to break concrete behind steel. Any worker can manually break the concrete, but it is necessary to choose workers who have done repair work before as they must be sensitive in breaking the concrete to avoid causing cracks to the adjacent concrete members.

8.2.2.2 Pneumatic Hammer Methods

These hammers work using compressed air; they weigh between 10 and 45 kg. If they are used on the roof or walls, their weight will be about 20 kg. They need an attached small power unit to do the job, but in large areas may require a separate, bigger air compressor, as shown in figure 8.2. This machine requires proper training for the worker that uses it. Compressed air hammers have a few initial costs. A few have been discussed by the Strategic Highway Research Program (SHRP) and, based on Vorster et al. (1992)—a research program of highways, the terms of the contract are governed by the contractor responsible for handling the breaking.

The use of pneumatic hammers is more economical when a small, rather than large, area is to be removed (see figure 8.3). A water gun is preferable for large areas (described in the following section). When the client does not specify the area that needs to be removed, the cost calculation will be based on square meters. In this



FIGURE 8.2 Air compressor.



FIGURE 8.3 Using a pneumatic hammer to remove wall concrete cover.

case, the risk will be low because the machine's initial cost is small, so a pneumatic hammer would be preferred. Some performance rates are about $0.025\text{--}0.25\text{ m}^3$ per hour using hammers weighing $10\text{--}45\text{ kg}$, respectively. In summary, pneumatic hammers are useful when small areas of concrete are to be removed.

8.2.3 WATER JET

This method has been commonly used since it was introduced to the market in the 1970s. It relies on the existence of water at the work site and on the removal of a suitable depth of concrete in a large area. It removes fragmented concrete, cleans steel bars, and removes part of the concrete behind the steel bars, as shown in figures 8.4, 8.5, and 8.6. The water jet is used manually by an experienced worker who has previously dealt with the hose, which is pushing water under high pressure

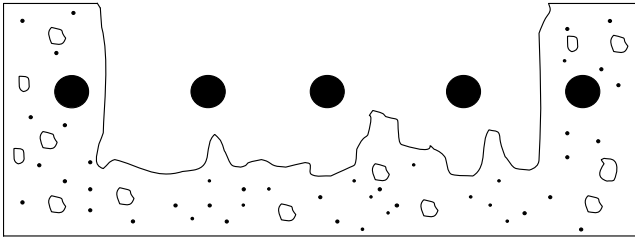


FIGURE 8.4 Shape of delaminated concrete when a water jet is used.



FIGURE 8.5 Concrete surface after using a water jet.



FIGURE 8.6 Surface after using a water jet.

or perhaps through the mechanical arm. Very high safety precautions need to be applied to the worker who uses it and the site around it.

The water used must not have any materials that affect the concrete, such as high chloride ions; in general, it must be potable water. The water gun consists of diesel engines and related through pressure pump and connected by hose that bears high water pressure from 300–700 kg/cm² at the nozzle and at least 400 kg/cm². At least 400 kg/cm² is required to cut the concrete and the rate of water consumption is about 50 L/minute. The performance rate of a water jet to break concrete is about 0.25 m³/hour—in the case of the use of a small pump—and can reach up to about 1 m³/hour if two pumps or one big pump is used.



FIGURE 8.7 Grinding machine.

8.2.4 GRINDING MACHINE

This is used to remove concrete cover in the case of large, flat surfaces. An example is a bridge deck, as in figure 8.7, but it must be done cautiously so that the process of breaking does not reach the steel. As in the case of any contact between the grinding machine and steel reinforcement, it will cut the steel bars and damage the machine. This method of breaking delaminated concrete is seldom used in the United Kingdom, because insulation film is often used to prevent the water, and it is used sparingly on U.S. bridges. The grinding machine is usually used after the water gun or the pneumatic hammer to obtain final concrete breakdown around and under the steel reinforcement. Therefore, one must take into account whether the thickness of the concrete cover is equal. The rate of removal of the concrete by this machine is very fast; it removes about $1 \text{ m}^3/\text{minute}$ and its cutting part is 2 m in width.

8.3 CLEAN CONCRETE SURFACES AND STEEL REINFORCEMENTS

This phase removes any remaining broken concrete with a process of cleaning. At the same time, the process of assessing the steel and cleaning up and removing corrosion from the roof takes place.

8.3.1 CONCRETE

The stage of preparing a surface by pouring the new concrete is one of the most important stages of the repair process. Before application of the primer coating, which provides the bond between the existing old concrete and the new concrete for repair, the concrete surface must be well prepared, and this takes place according to the materials used. In all cases, the concrete surface must be clean and not contain any oils, broken concrete, soil, or lubricants. If any of these elements is present, the surface must be cleaned completely through sand blasting, water, or manually using brushes. This stage is very important and very necessary, regardless of the type of material used to bond the new concrete with the old. Neglecting the preparation of the surface might affect the repair process as a whole because this stage of repair is the less expensive stage in the repair process.

It is not necessary to prepare a surface when a water gun is used to remove the delaminated concrete because the surface will be wet enough and will be clean after the crushing process. One of the benefits of using a water gun is that air pressure has cleaned the surface of concrete and removed any growths on the fragmented concrete, thus supplementing the creation of the surface material to use for the link with new concrete repair.

When cement mortar or concrete is used in the repair, the concrete surface will be sprayed by water until saturation can be reached. The stage of saturation can be reached by spraying water on the surface for 24 hours or through wetting burlap. The water spray must be stopped and the burlap removed for about 1–2 hours, depending on weather conditions, until the surface is dry. It is then coated by a mix of water and cement only, which is called slurry and is applied by brush. Epoxy coating can also be used as an adhesive between new and old concrete; it is necessary to follow the manufacturer's specifications and warnings.

The preparation of the concrete surface to achieve a better bond between the old and new concrete is stated in the American specification ACI (American Concrete Institute) 503.2-79. This specification says that the preparation of surfaces to receive epoxy compound applications must be given careful attention as the bonding capability of a properly selected epoxy for a given application is primarily dependent on proper surface preparation. Concrete surfaces to which epoxies are to be applied must be newly exposed, clean concrete free of loose and unsound materials. All surfaces must be meticulously cleaned, be as dry as possible, and be at proper surface temperature at the time of epoxy application.

ASTM C881-78 is specific about the epoxy material that performs the bond; it must be well defined by supplier specifications identical to the different circumstances surrounding the project, particularly when, with an increase of temperature, change might occur in a resin, as well as the nature of loads carried by the member requiring repair. It is worth noting that during the execution of the repair process, the use of epoxies will be significant. Therefore, we must take into account the safety factor for workers who use such material. These workers must wear their personal protective equipment (PPE), such as gloves and special glasses, for safety issues because these materials are very harmful to skin; epoxy can cause many dangerous diseases if one is exposed to it for a long time. Also, some epoxies are flammable with temperature, and that fact must be taken into account during storage and operation.

8.3.2 CLEAN STEEL REINFORCEMENT BARS

After removal of the concrete covers and cleaning the surface, the next step is to evaluate the steel reinforcement by measuring steel diameter. If the cross-sectional area of the steel bars is found to have a reduction equal to or more than 20%, additional reinforcing steel bars must be added. Before pouring new concrete, one must be sure that the development length between the new bars and the old steel bars is enough, as shown in [figure 8.8](#). It is usually preferable to link the steel by drilling new holes in the concrete and connecting the additional steel on concrete by putting the steel bars in the drilled hole filled with epoxy. However, in most cases the steel bars are completely corroded and need to be replaced.

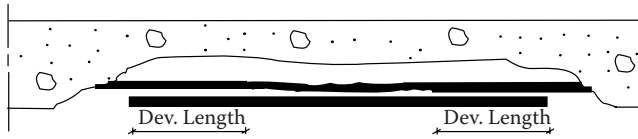


FIGURE 8.8 Installing additional steel.

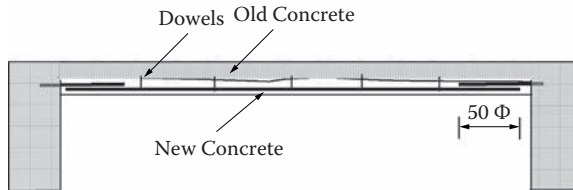


FIGURE 8.9 Steel reinforcement installation for slab repair.

In the case of beams and slabs that need to add additional steel reinforcement bars, it is preferable to connect the steel bars with concrete by drilling new holes in the concrete and making the bond of the steel bars in the holes by using adhesive epoxies. For beam repairs, the additional steel bars are fixed in a column that supports this beam. In the case of slabs, the steel bars are fixed in the sides of the beam that is supporting the slab, as shown in figure 8.9. The dowelling will be fixed in the beam side by drilling holes to a depth of around 70–80 mm; the dowel will be fixed in the holes by epoxy.

The depth of the hole may differ, depending on the type of the epoxy and the supplier's assumptions and recommendations. The hole's diameter will be 40 mm greater than the bar's diameter to ensure that the steel bars are fixed completely. It is always 12 or 16 mm; it is preferable to increase the bar's diameter and decrease its number to reduce the number of holes in the beams. The dowel is put in the drilled hole, as shown in figure 8.9, with a length 50 times the bar's diameter, which overlaps with the new steel bars and is fixed between them by the steel wire or welding. The bars are fixed in the two directions by putting a small dowel from the slab in the intersection between the steel bars in the two directions; these small dowels are fixed to the slab by epoxies, also.

Most coastal cities have special creations by architectural firms and design is based on placing all the balconies facing the coast to view the sea and to increase the value of the units in the building. At the same time, balconies require distinctive structural elements in their design and execution. From a structural point of view, a balcony is cantilevered; the cantilever has the lowest redundancy of all structural elements, so it needs special attention in the case of repair and restoration.

In practice, balconies often have corrosion on the upper steel reinforcement, as it is the main steel, particularly in the cantilever. The tiles will be removed, the concrete cover removed, and then the reduction on the upper steel reinforcement will be inspected, as we mentioned earlier. If the condition of steel bars is bad and it is necessary to add new upper steel reinforcement, the upper steel bars must extend on the adjacent span by length equal to 1.5 times the cantilever length, as in figure 8.10.

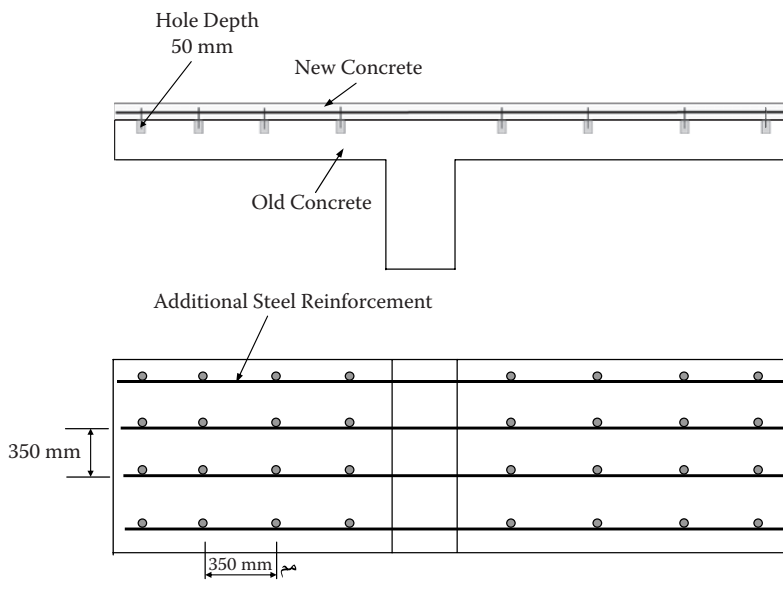


FIGURE 8.10 Cantilever repair.

The dowels will be installed vertically, with a distance of about 350 mm between them. They will be fixed by the concrete slab with epoxies to a depth of 50 mm and the new main steel bars are attached by it.

It is worth mentioning that, in all repair processes—and especially in cantilevers because they are a point of weakness in terms of structure safety—one must pay attention to the design and execution of the wood form and temporary supports, as well as the part of the building that will be overloaded during the repair process. It is necessary to consider the structural system and stress distribution in different elements, the weak points in the building, the building's maturity, and the method of repair that will be applied.

The work of repairing the beams is almost the same as for the slab repair process. The difference is in fixing the dowels in the columns, as shown in figure 8.11, and fixing the stirrups through making holes with depth of around 50–70 mm (according to the epoxy manufacturer's recommendation). Epoxy is on both sides of the beam to stabilize stirrups, about 8 mm/200 mm, as shown in figure 8.11. After that, as a normal procedure, epoxy coating is applied as an adhesion between old and new concrete and then the new concrete is poured manually or by shotcreting. As shown in the figure, the beam is strengthened by increasing the cross-sectional dimension and adding steel reinforcement.

As for the columns, the repair is performed as shown in figure 8.14, which shows the casting on-site; however, the minimum distance allowed to cast concrete easily is about 100–120 mm from each side. The dowel is fixed for the first floor, as in figure 8.12; this does not require drilling holes in the base, but rather making the legs to steel bars to rest on the foundation. The steel bars will be distributed around the column circumference as in the figure to cope with the reduction of steel reinforcement

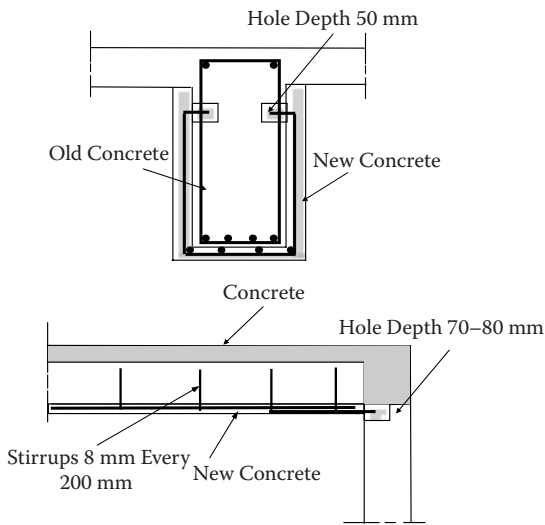


FIGURE 8.11 Beam repair.

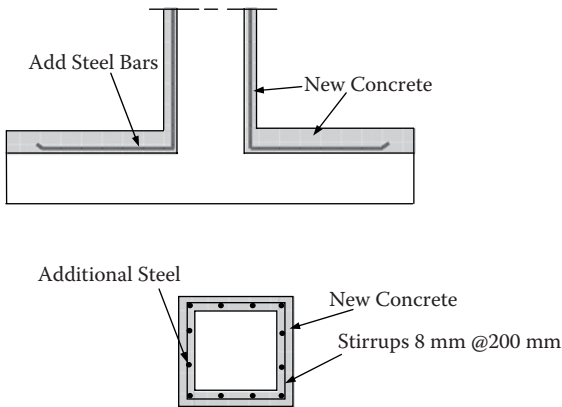


FIGURE 8.12 Repairing a concrete column.

cross-sectional area due to corrosion. The steel cross-sectional area percentage to the concrete column cross-sectional area is the same as that in the old column member and not less than as stated in the code.

Note that when a water jet is used, it will clean and remove corrosion from the steel; if it is not used, sand blasting can be used to remove corrosion. Then, the old steel can be painted by using epoxies after cleaning the steel bars completely, especially from the effects of chlorides. Slurry (mix between cement and water) has been used. In the case of using mortar cement in the process of repair, that situation has not used painting steel by epoxy to obtain the benefit for alkaline protection from the cement mortar to the steel reinforcement. Note that the improved cement slurry can be dried quickly, so it will be ineffective in repairs that require the installation of a form after painting the bars. However, it works well in cases where the time between

painting steel and pouring cement mortar is short; this period of time should not exceed 15 minutes, as stated by a U.S. Army manual in 1995 (Department of the Army, U.S. Army Corps of Engineers 1995).

8.4 NEW PATCH CONCRETE

Some mixtures available on the market have been mixed in a certain way and they can be used easily in repairs for small areas, as this mixture is very expensive. In the case of large surface areas to be repaired, the mixture preparation and mixing will be on-site to be less costly, but this must be carried out by an expert. In addition to these properties of concrete characteristics, using a pump for pouring or shotcrete needs a special design mix.

In the United States and Canada, most of the contractors who work on bridge repair use their own concrete mix, which has secret mixing proportions based on the available materials in local markets. The ready-mix concrete factories can provide guarantees in case of corrosion due to carbonation, but if it is due to chlorides, they do not provide a guarantee for fear of the presence of chloride after the repair process is complete. Most manufacturers of materials used in mixing are field execution contractors, but when they supply materials only, they will provide all the information and technical recommendations for the execution, performance rates to calculate the required amount of the materials, and appropriate method of operation.

Worldwide, all companies operating in the construction field and dealing with chemicals must have a competent technical staff that can assist on-site personnel or supervise workers to avoid any errors and to define responsibility if defects are found. Two types of materials are used as a new mortar for repair: polymer mortar or cement mortar enhanced by polymer. Both of them will be described in the following sections.

8.4.1 POLYMER MORTAR

This mortar is ready to give the specific components of repair and what is required from the ability to control the ease of operation and the influence as well as what can be controlled of the setting time. It provides a good bond with the existing old concrete, so it does not need any other to develop this cohesion. This mortar has high compression strength of around 50–100 MPa and high tensile strength, also.

Despite the many advantages of such mortar, its properties are different from those of the existing concrete as the polymers have a coefficient of thermal expansion equal to $65 \times 10^{-6} ^\circ\text{C}$, corresponding to a concrete thermal expansion coefficient equal to $12 \times 10^{-6} ^\circ\text{C}$. Moreover, the modulus of elasticity for mortar is much less than in the case of concrete. These differences in the properties lead to the presence of cracks as a result of generating internal stresses; however, they can be overcome by aggregate sieve grading to make the proportion of polymers less, which reduces the difference in the natural properties. There are many tests to define this mortar and noncompliance with the concrete (e.g., ASTM C 884-92: “Standard Test Method for PCBs: Comparability between Concrete and Epoxy-Resin Overlay”).

8.4.2 CEMENT MORTAR

Polymer mortar gives physical protection to the steel reinforcement; however, cement mortar gives passive protection as it increases the alkalinity around the reinforcement. The trend now is to make cement mortar for optimal use in the case of repair as a result of corrosion because it has the same properties as the existing concrete. In addition, it gives passive protection to steel, the basis for the protection from corrosion.

Some polymers are added to the cement mortar through liquid or powder to improve some of the mortar's properties (ACI Committee 548 1994; Ohama 1995) and to increase flexural resistance and elongation, reduce water permeability, increase the bond between old and new concrete, and increase the effectiveness of its operation. The polymer used is identical to the cement mortar mixture; the properties that control the polymers in the cement mortar are stated in the ASTM C 1059-91 specification for latex agents for bonding fresh to hardened concrete. Silica fume can be used with the mixture, as well as superplasticizers to improve the properties of the used mortar and reduce shrinkage.

8.5 EXECUTION METHODS

There are several ways to implement the repair process, which is entirely dependent on the type of structure member to be repaired and the materials used in the repair. These methods are described in the following sections.

8.5.1 MANUAL METHOD

The manual method is used in most cases, especially when small spaces are to be repaired (fig. 8.13). A wooden form can be made and then concrete can be poured



FIGURE 8.13 Manual method for repair.

into the damaged part, as in figure 8.14, in the case of concrete columns or vertical walls. One must take into account an appropriate distance to cast the concrete in the wooden form easily. This method is commonly used for its efficiency and has little cost. It does not require expensive equipment, but extensive experience in fabrication, installing the wooden form, and the casting procedure is needed.

8.5.2 GROUTED PREPLACED AGGREGATE

As shown in figure 8.15, the aggregate should be placed with gap grading in the area to be repaired. The next step is to make a grouting fluid by injecting it inside the

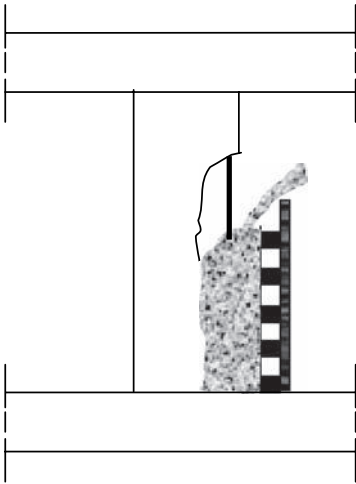


FIGURE 8.14 Casting concrete on-site.

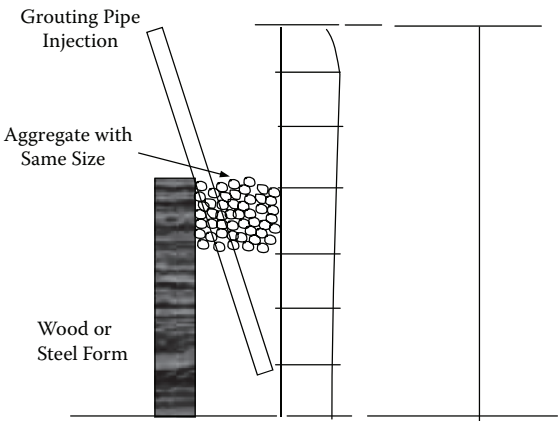


FIGURE 8.15 Injected preplaced aggregates.



FIGURE 8.16 Using shotcrete.

aggregate by a pipe with pump to fill the gap between the aggregate and the grout. This method of repair is used in repairing bridge supports and other special applications. This method requires special equipment such as the pipe injection, pump, and other special miscellaneous equipment. Therefore, one can conclude that this method is used by private companies with high potential.

8.5.3 SHOTCRETE

Shotcrete is used when large surfaces need to be repaired, but the mix and components of concrete suitable for the use in shotcrete need special additives and specifications. As shown in figure 8.16, health and safety precautions must be carefully followed for the worker using shotcrete, as it contains polymers and special additives. In the concrete mix design, the nominal maximum coarse aggregate size must be defined to be suitable to the shotcrete equipment's nozzle and pump to avoid any problem during concrete casting. Complete member casting is used when total reconstruction is required for the concrete member whose steel reinforcement bars have been depleted due to corrosion. Therefore, it is necessary to pour concrete for the complete member with full depth, as shown in figure 8.17.

Frequently, in practice (e.g., when bathrooms are repaired), the concrete slab will be in very bad condition, so the ordinary repair procedure will not be efficient. In this case, the entire concrete slab will be demolished, a new steel reinforcement will be installed, and the concrete slab will be poured, as shown in figure 8.17. A decision to use this method must take into consideration the whole building's condition. This method is usually easy to apply to the bathroom slab because, in residential buildings, it is usually designed as simply supported as it drops around 100 mm for plumbing pipes.

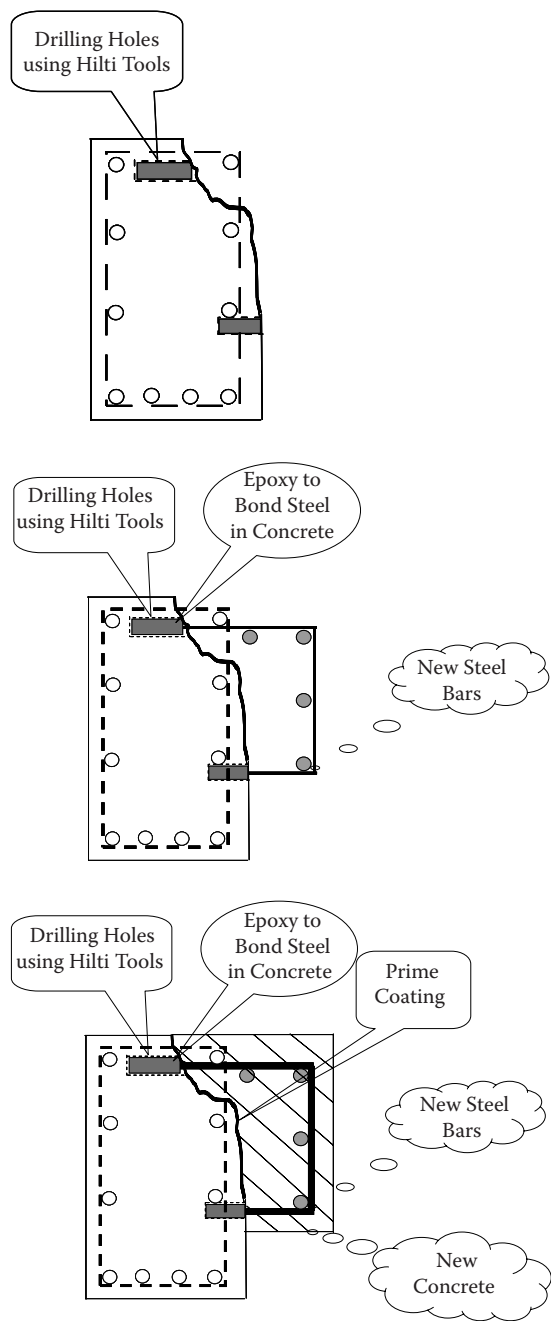


FIGURE 8.17 Steps of repair.

8.6 REPAIR STEPS

Figure 8.17 presents a case study for repairing a reinforced concrete ring beam supporting steel tanks containing oil, as in the case study in chapter 4. The ring beam is structurally carrying a tensile strength due to an earth load affecting the beam, which puts the ring beam under tension. Corrosion of the steel reinforcement bars means reduction of the capacity of the ring beam. Thus, the best solution is to increase the steel bars to overcome the reduction on the steel cross-sectional area. The process of repair takes place in three steps. In step 1, holes are drilled and steel stirrups are put in the holes and fixed by epoxy. In step 2, the main steel bars are installed and the concrete surface is coated by epoxy so that the existing old concrete will bond with the new concrete. In step 3, the new concrete is poured with a proper mixing design, which is improved by additives such as polymers.

8.7 NEW METHODS FOR STRENGTHENING CONCRETE STRUCTURES

There is another way to strengthen reinforced concrete structures, including traditional methods such as the use of steel sections. This method has many advantages, including the fact that the thickness of the concrete sections need not be increased significantly. This is important to maintain the building's architectural design. On the other hand, it is a quick solution to strengthen the concrete member and is usually used in industrial structures and buildings. When the cause of strengthening results in corrosion in steel reinforcement, the solution of placing steel supports is not considered appropriate but can be used, taking into account that it requires epoxy paints periodically.

Recently, many studies have been conducted on using fiber reinforcement polymer, which has many advantages over steel. Most importantly, it does not corrode and therefore can be used in any environment exposed to corrosion. There are some modern ways to protect steel reinforcements from corrosion, such as replacing steel reinforcement bars with bars from fiber polymer and also using some epoxies that coat the concrete surface by painting.

Generally, the goal of strengthening the structure is to restore the concrete member to be able to withstand loads, to increase its ability to carry loads greater than the loads for which it was designed, or to reduce the deflection that occurs when the concrete member is overloaded. The strengthening can proceed as a result of the presence of cracks and needs to stop the increase of cracks or reduce crack width in the concrete member. Therefore, strengthening is required in the following cases:

- An error in the design or execution causes a reduction in the steel cross-sectional area or a decrease in concrete section dimensions.
- Deterioration in the state of a structure leads to weakness of the resistance (an explicit example of this is the corrosion in steel reinforcement).

- An increase in loads increases the number of stories or changes the building's function (e.g., from residential building to office building).
- The structural system is changed (e.g., removing walls).
- Holes are made on the slab, reducing slab strength.
- Some special structure needs strengthening—for example, bridges: As the flow of traffic increases with time, the flow of traffic and heavier trucks may be higher than what it was designed for, so the bridge needs to be strengthened to accommodate the overloaded and new operation mode.
- The load may change with time, such as a building in an earthquake zone. This happened in Egypt when the earthquake map was changed and, consequently, some buildings needed to be strengthened to match the new specifications.
- The structure may be exposed to high temperature due to fire.
- A big machine may need to be installed in an industrial structure.
- In industrial buildings, the machines mature over time or change mode of operation, resulting in high vibration, so the structure needs to be strengthened.

8.8 USING STEEL SECTIONS

Using steel sections in the large scale to strengthen the reinforced concrete structure has different advantages. The method of strengthening the concrete member varies depending on the member that needs to be strengthened. The strengthening process is calculated based on the reduction in strength in the concrete section and this reduction is compensated by using steel sections.

If a concrete beam needs to be strengthened due to a lack of steel reinforcement, a U-shape can be used (figure 8.18) when a big change in architectural view is not warranted and there is minor reduction in member capacity. On the other hand, in the case of a significant reduction in concrete member capacity and a need to increase the depth significantly, an I-beam section will be used (figure 8.18). Another way to strengthen beams is by stabilizing sheets of steel (figure 8.19) and fixing by mechanical bolts or

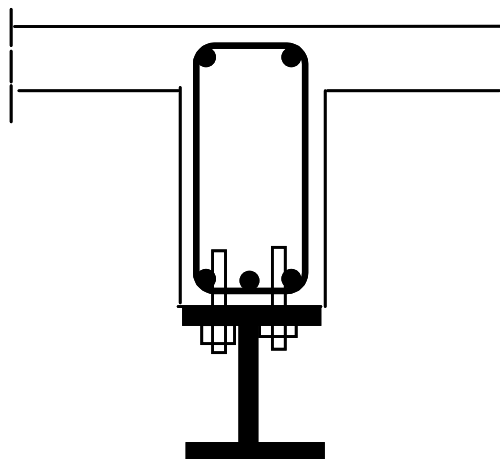


FIGURE 8.18 Beam strengthening.

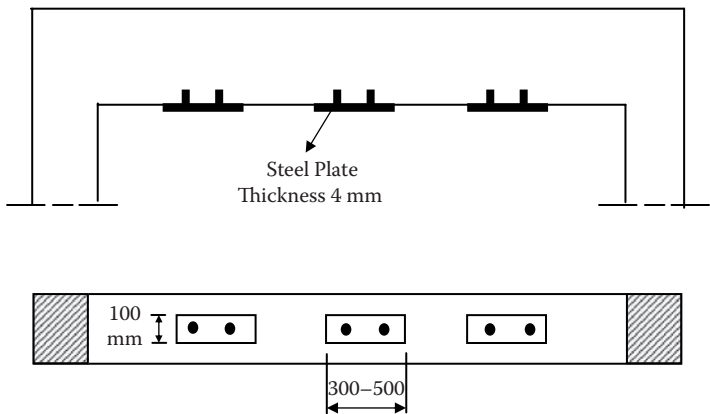


FIGURE 8.19 Strengthening a concrete beam by steel plate.

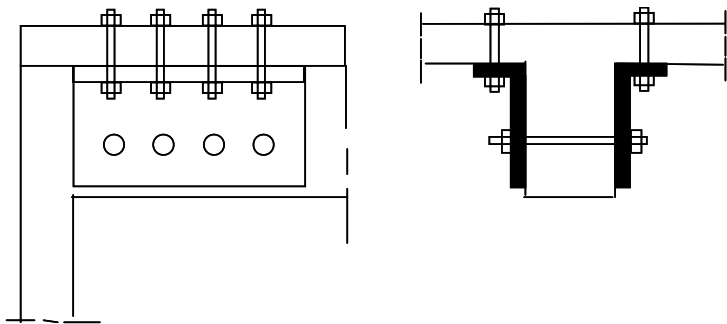


FIGURE 8.20 Beam strengthening in direction of shear.

through chemical substances such as epoxy. The purpose of adding this steel sheet is to increase the moment of inertia of the beam and thereby increase its ability to withstand stress greater than the flexural stresses for which it was designed. In the case of beams that have a problem in shearing force direction, the beam can be strengthened (fig. 8.20).

Cracks present on the upper surface of the slab in the beam direction indicate a shortage in the upper reinforcement of the slab; it can be strengthened by using steel sheets on the upper surface, as shown in figure 8.21. These plates are fixed by bolts with nuts, which will be fixing from the lower direction of the slab, as in the figure. When the flat slab needs to be strengthened, the procedure is performed as shown in figure 8.22; the steel angles will be fixed between slab and columns. Short cantilevers are used in the frames in factories or bridges; the main high stresses are due to shear stresses and the other stresses are due to flexural stress. To strengthen the cantilever, as shown in figure 8.23, a steel sheet should be added at its two sides and compression force applied with bolts, thereby reducing the likelihood of a collapse due to shear.

The most common situation is to strengthen the columns, and the solution is in figure 8.24. This is an appropriate practical solution as it does not increase the column dimensions significantly with increasing column capacity. This method is summarized

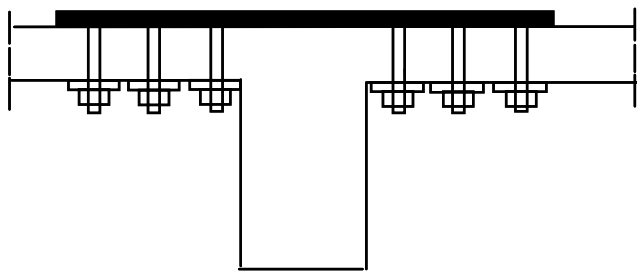


FIGURE 8.21 Slab strengthening in direction of upper steel.

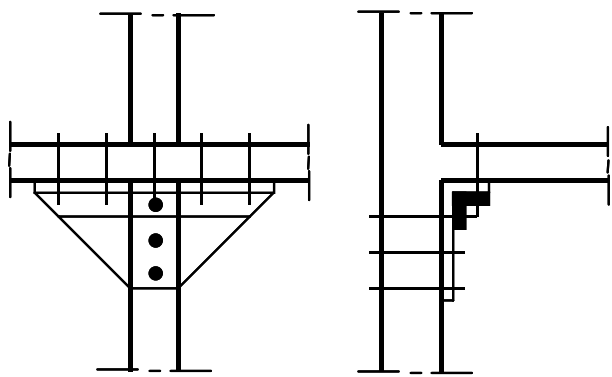


FIGURE 8.22 Strengthening the flat slab.

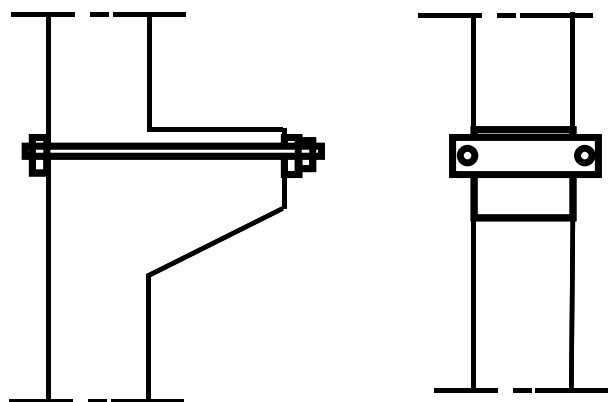


FIGURE 8.23 Strengthening short cantilevers.

by installing in the four corners of the columns four equal steel angles with 50 mm and thickness of 5 mm; they are kept close by sheets across the angles of 50 mm \times 5 mm with sheet plates that are 500 mm wide and have a thickness of 5 mm. These sheets will surround the column, welding to the steel angles every 200 mm as shown in the figure. The steel angles should be fixed to the upper and lower slabs by the upper and lower four angles around the column; these angles are 70 mm wide and have a thickness of 7 mm.

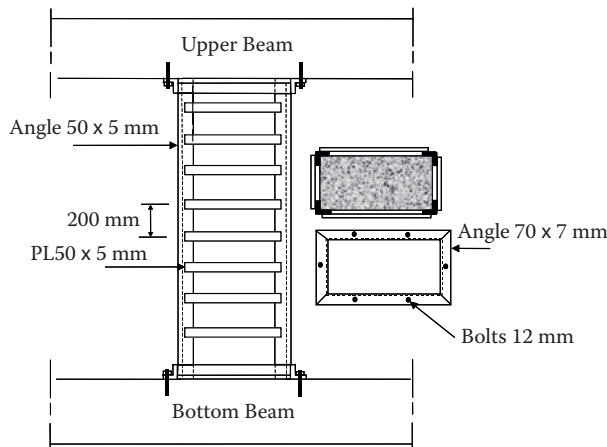


FIGURE 8.24 Strengthening a reinforced concrete column.

The fixation of the upper and lower angles to the slab or beams by mechanical connector or using certain types of epoxies designed by the bolt factory follows manufacturer recommendations. It is important to know that these bolts transfer the load from up to down. In previous strengthening methods, the steel section is selected with dimensions as small as possible so as to maintain the architectural view; these can be covered by using wire steel mesh and then applying plastering, covering the surface by wood, or using special plastering or decoration methods to provide an acceptable view.

8.9 FIBER-REINFORCED POLYMER (FRP)

Modern science does not stop in the construction field; as we know, developments in computer technology occur continuously, rapidly, and in all areas, including engineering constructions. At the same time, there has been an evolution in some materials, such as plastic, based on epoxies, as explained in previous chapters. The use of such epoxies increases the quality of concrete and the concrete compressive strength can reach 1500 kg/cm^2 .

The development of competition between international companies led to the creation of methods for strengthening reinforced concrete structures. The traditional ways of strengthening these structures have been by increasing the concrete sections or through the steel sections, including through strengthening the members of various constructions or strengthening bridges and using steel sheets kept in the concrete through bolts. From the fundamental disadvantages for strength the concrete member by the steel sheets is the corrosion of the steel sections. Therefore, it needs to be protected from corrosion; particularly, protection is necessary in the region between the interacting surfaces of concrete with steel. This is often the subject of using a steel section to strengthen the concrete member in corrosive environmental conditions; eventually, however, use of steel for strengthening as a practical solution is not ideal in that situation. On another level, the steel has great weight, which increases the weight on the concrete member and the difficulty of strengthening due to installation of steel

sheets on the concrete as a result of the weight of the steel sheet, as well as the difficulty of transportation, storage, and stabilization.

The growing interest in FRP systems for strengthening and retrofitting can be attributed to many factors. Although the fibers and resins used in such systems are relatively expensive compared with traditional strengthening materials like concrete and steel, labor and equipment costs to install FRP systems are often lower. Fiber-reinforced polymer systems can also be used in areas with limited access, where traditional techniques would be very impractical—for example, a slab shielded by pipe and conduit. These systems can have lower life-cycle costs than conventional strengthening techniques because the FRP system is less prone to corrosion.

The presence of such defects in the evolution of this method with other materials has led to resorting to the use of plastic fiber bars. Fiber-reinforced polymer (FRP) can serve as an alternative to the use of steel sheets. The use of FRP has a wide range of advantages and offers an alternative to the steel used in the strengthening process. Its distinctive characteristics include its inability to cause corrosion, which operates under any environmental circumstances, caution as the concrete member to be connected to the heat and electricity, resistant to chemicals, which makes it a solution suitable for industrial structures. There are different types of FRPs; the famous type is carbon fiber–reinforced polymer (CFRP), which is most commonly used as appropriate in practical applications and because of its unique properties in terms of resistance and the resistance with time, as well as resistance to stress.

8.9.1 CFRP

In the market there are commercial dimensions of CFRP (commonly 1–1.5 mm, with 50–150 mm width). All of these types consist of 60–70% carbon fiber in one direction with diameter of about 10 μm embedded in epoxy resin. The mechanical properties of CFRP differ from one type to another, but generally the modulus of elasticity has a value between 165 and 300 N/mm^2 and the tensile strength has a value range of 2800 N/mm^2 to a lower value of 1300 N/mm^2 . We find that the lower value of the modulus of elasticity corresponds to the maximum tensile strength value.

The carbon fiber sheet is shown in figure 8.25; this sheet is embedded in epoxy resin. The important property is the density, which is about 1.6 t/m^3 compared to

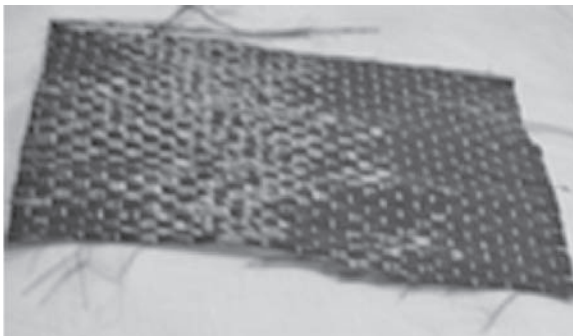


FIGURE 8.25 Carbon fiber sheet.

steel's density of 7.8 t/m^3 . This is a big difference in that weight less than the weight of steel by about 10 times makes the transfer and is easily fixed. The adhesive material used in the sheets of carbon fiber resin is a mixture of high resistance and the rule of filling quartz; its tensile strength is about 30 MPa, up from the concrete tensile strength by about 10 times. Also, the rate of shrinkage and creep is small and can bear high temperatures and exposure to chemicals. The mechanical properties of carbon fiber sheets in the longitudinal direction are often controlled by the fiber and its behavior is linear elastic until collapse.

8.9.2 APPLICATION ON-SITE

Before starting to work to strengthen a concrete structure, it is necessary to examine fully all the circumstances surrounding the structure in order to determine that strengthening the structural work is important and that the company supplying the plastic fiber is preferred to perform the installation because highly experienced personnel are needed to design and identify segments of the required thickness. If strengthening is needed as a result of steel corrosion or due to chemical attack, it is important and necessary for the success of the strengthening process to remove rust from steel bars and chemical substances that attack the concrete before applying the strengthening procedure.

The concrete surface must be settled through sand blasting to level the surface, eliminating any gaps or delaminated concrete. After sand blasting, the next step is to collect all the damaged concrete and sand by air suction, air pressure, or water pressure to clean the surface. Before fixing the CFRP, it is strongly recommended that the surface be level and clean; in the case of any cracks or delaminated concrete, the surface must be repaired before fixing the CFRP.

The mixture that provides the bond between the segments of CFRP and the concrete surface must be prepared. This mixture is very important in the repair as it is responsible for transferring the stresses from the concrete to CFRP. Therefore, the preparation of the epoxy mix and the percentage of mixture components must be precise and under supervision of the manufacturer because much experience in application, transportation, and storage is required from technical and safety points of view. These epoxy materials are a blend of resin (a) and solvent (b) and are mixed by an electric hand mixer for a period of about 3 minutes, until the mixture homogeneity find clear if the mixing is near the bottom and the sides of the container, it will be slower and generally greater speed allowed is about 500 rpm.

When the epoxy is applied to the surface, its thickness is about 0.5–2 mm; it must be applied accurately, making sure that all the gaps have been filled and there are no

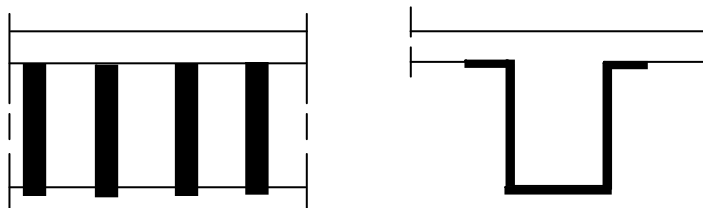


FIGURE 8.26 Strengthening beam for shear failure.



FIGURE 8.27 Column repair.

air voids. The images in figures 8.26 and 8.27 show how to install the CFRP beam or a roof of reinforced concrete installed on a reinforced concrete column. The repair of the beam due to overshear stress or reduction on shear stress capacity is shown in figure 8.26 and repair of the concrete column by CFRP is shown in figure 8.27.

8.10 GENERAL PRECAUTIONS

Note that the safety of a structure relies primarily on the quality of the repair process in terms of execution, design, or planning. As we mentioned earlier, the repair process, identifying the necessary temporary support of the building, and the location depend on the nature of a structure and its structural system. Also, determining the method of breaking the defective concrete and the way of removing it, as well as choosing the materials that will be used in the repair, must be in full conformity with the state of the structure and nature of the site of the structure, or location of the retrofit members within the overall structural system. All these elements require special expertise. Without these experiences in the repair process, the possibility of risk in terms of structural safety is very high. Also, an ineffective repair process has serious economic impact.

From the economic point of view, it must be remembered that the repair process in general is high in cost in terms of materials used and the precautions that must be taken during execution, or in terms of trained workers who must carry out the repair process. Building safety and the health and safety of the workers are crucial; often epoxy materials or polymers can cause breathing and skin problems with time. Unfortunately, we find that health and safety rules are of great interest in developed countries, but do not receive the same degree of attention in developing countries—despite the fact that the interest in this matter provides a lot of time and money for the whole project and represents a great danger to the management of the project.

It is also important not to forget that after equipment has been used, it must be cleaned very well. Leaving equipment uncleaned after finishing the work will be a main reason for operational problems in the future. This is due to using epoxies,

polymers, and other materials that are problematic for machines if they are not cleaned well.

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9 Risk-Based Maintenance Strategy

9.1 INTRODUCTION

In general, the economic factor is one of the most influential in an engineering project. The economic cost of repairing the concrete structure, as well as the selection of a suitable protection system to protect the steel reinforcement from corrosion, is one of the major factors influencing the choice among various alternatives for repair or protection. Virtually every method has an expected lifetime. By knowing structure lifetime, one can easily calculate the number of expected maintenance projects to be done throughout that time. For a new structure, it is important to conduct an economic study to choose among alternatives for structure protection by considering the lifetime of the protection method with respect to the structure's lifetime. In addition, it is necessary to take into account the initial costs of periodic maintenance.

Generally, the cost calculation is based on a summation of the initial costs of protection with the cost of maintenance that will be performed in different periods and periodically. The number of maintenance times during the lifetime of a structure varies depending on the method of protection as well as the method used in the repair procedure during maintenance. On the other hand, when one wants to perform maintenance to restore an existing older structure to its original strength, the method of rehabilitation is also governed by the initial costs of repair, as well as the number of times the maintenance will be done and its costs over the remaining lifetime of the structure.

The previous chapters discussed how to choose among the various alternatives for protection methods as well as the appropriate repair methods and the materials usually used in repairs. The previous discussion was from a technical point of view. However, in this chapter, the way of comparing between different alternatives will be discussed from an economic point of view to assist in decision-making procedures. Therefore, the appropriate method will be used to compare alternatives to clarify the economic factors that affect the method of calculation.

As we have stated before, the lifetime of maintenance or protection of a structure must be taken into consideration, so the required time to perform the maintenance will be discussed clearly as the first principle to calculate this time. A practical example illustrating the comparison between different alternatives will also be applied. In this example, differences between the methods of protection for different structural economies will be detailed.

The process of determining the time of maintenance depends on the maintenance cost estimate versus the probability of structure failure; therefore, it is important to decide on selection of the appropriate time to perform maintenance. The method

of decision making will be clarified, as well as how to use it from the standpoint of determining the right time for maintenance in a less expensive verification process called “optimization procedure.”

9.2 BASIC RULES OF COST CALCULATION

There are several basic rules for calculating the economic costs of any engineering project, as well as for applying those rules to select the type of protection and method of repair for a structure. The most popular methods of economic analysis tools are the present value, future value, and interest rate of return. Here, the method of calculating the present value is described briefly as it is the easiest way to select the appropriate method of economic repair and an appropriate system to protect the structure from the effects of corrosion.

9.2.1 PRESENT VALUE METHOD

The cost of protecting reinforced concrete from corrosion consists of the preliminary costs of the method of protection and the money paid with the beginning of construction. On the other hand, the cost of maintenance and repair will be over the lifetime of the structure. In many cases, the cumulative cost of maintenance and repair is higher than initial costs. In many projects, cost calculation is often based on the initial costs only, but the result is that the total cost is very high compared to the structure's cost estimate, which is only the initial cost.

A method of present value is used to calculate the present value of future repair, including the cost of equivalent current value with the assumption that the repair will take place after a number of years, n :

$$\text{Present value} = \text{repair cost} (1 + m)^{-n} \quad (9.1)$$

where m is the discount rate, which is the interest rate before inflation. For example, assuming that the interest rate is 10% and the inflation rate is 6%, the discount rate m is equal to 4%, or 0.04.

The entire structural cost consists of the initial cost, which is called capital cost (CAPEX), and the sum of the present values of future costs due to maintenance, which is called operating cost (OPEX). When the rate of inflation increases, the cost of future repair will not be affected. But when it decreases, the present value of future repair will increase. In this chapter, we will assume an inflation rate of about 4% that depends on a country's general economy. Every country has its own published inflation rate.

9.2.2 REPAIR TIME

The time required to repair the structure as a result of steel corrosion is the time at which corrosion begins in the steel reinforcement bars and the time needed to spall the concrete cover with signs of concrete deterioration. This essentially requires work with repair. Tutti (1982) gave a simple explanation of a process of corrosion with time; the steps were for all types of corrosion and there was no difference if

corrosion happened as result of chloride attack or carbonation propagation. However, the invasion of chlorides, as well as the carbonation of concrete, takes a long time to break the passive protection layer on the steel bars and start corrosion.

After that, from the beginning of corrosion to a significant deterioration in the concrete, when repair will be necessary, will take more time. Raupach (1996) pointed out that in the case of concrete bridges, degradation occurs in about 2–5 years. Therefore, the time for repair is the total time required for the protection of depassivation in addition to 3 years.

The steps of corrosion's effect on a concrete structure are illustrated in figure 9.1. Note that after construction it will take time until chloride concentration or carbonation accumulates on a structure's surface and then spreads into the concrete, as shown in the figure. The next step is the propagation of chlorides or carbonation until the steel bars are reached. The third step is the start of corrosion on the steel bar, which at this time will have an impact on the concrete strength by reducing steel diameter and cracks occurring on the concrete surface. The last step is an increase in crack width until spalling of the concrete cover.

From the preceding analysis, we find that the time required for repair depends on the time needed to increase the percentage of chloride concentration to the limit that will initiate the corrosion, in addition to the rate of corrosion, which will happen after that. The previous chapter discussed several methods for protecting the structure from corrosion. These different methods delay the start of corrosion for a longer period of time as they reduce the rate of the chloride or carbonation propagation in concrete. They will reduce the rate of corrosion after that as well. Note that the preceding analysis relies on the noninterference of the hair cracks on the concrete in the rates of spread of chlorides or carbonation within the concrete. It is assumed that the design was based on the absence of an increase in the cracks more than that permissible in codes (as in chapter 5), as well as that the concrete was produced based on a quality control procedure according to code; thus, the presence of cracks is assumed to be within allowable limits.

The time required to start the structural repair depends on the nature of the surrounding weather and environmental factors that affect the beginning, as well as the rate, of corrosion. This time is determined by knowing the rate of corrosion and the required time to spall the concrete cover. The deterioration of concrete increases the

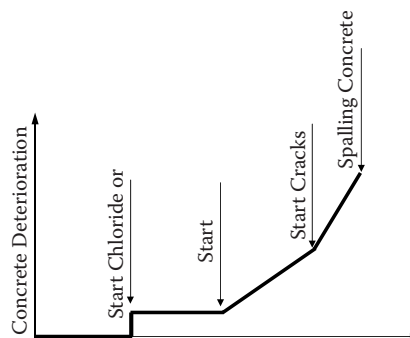


FIGURE 9.1 Sketch represents concrete structure deterioration process.

probability of a structure’s collapse with time; some studies have identified the probability of failure, which should not be beyond the structural reliability classified in various specifications.

A study on residential buildings by El-Reedy and Ahmed (1998) focused on how to determine the appropriate time to perform repair due to corrosion on the concrete columns. It considered environmental conditions around the structures: They were affected by humidity and temperature, which have a large impact on the increasing rates of corrosion. A corrosion rate of 0.064 mm/year reflects dry air, while a rate of 0.114 mm/year is based on a very high moisture rate. This study took into account the increasing resistance of the concrete over time, as well as the method of determination in the case of higher steel or low steel columns, with different times required for the repair process.

9.2.3 CAPACITY LOSS IN REINFORCED CONCRETE SECTIONS

According to the fundamentals of design of any reinforced concrete member, the member’s capacity depends on the cross-sectional dimensions (concrete and steel area) and material strength (concrete strength and steel yield strength). In the case of uniform corrosion, as shown in figure 9.2, the total longitudinal reinforcement area can be expressed as a function of time, t , as follows:

$$As(t) = \begin{cases} n\pi D^2 / 4.....for..t \leq T_i \\ n\pi \left[D - 2C_r \left(t - T_i \right) \right]^2 / 4.....for..t > T_i \end{cases} \tag{9.2}$$

where

- D = the diameter of the bar
- n = number of bars
- T_i = time of corrosion initiation
- C_r = rate of corrosion

This equation takes into account the uniform corrosion propagation process from all sides.

The curves in figures 9.3 and 9.4 show that, over time, the collapse of a structure is more likely with the increase in the rate of corrosion. The El-Reedy and Ahmed

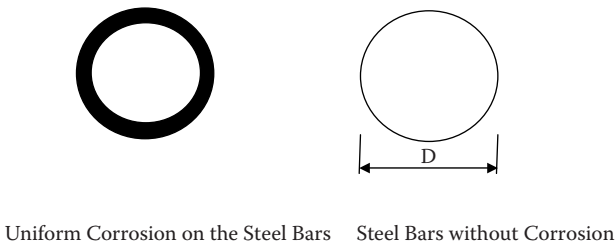


FIGURE 9.2 Reduction in steel diameter due to uniform corrosion.

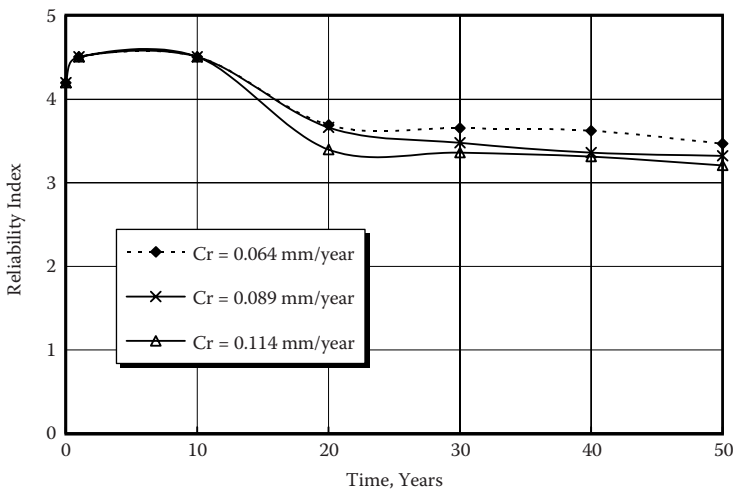


FIGURE 9.3 Effects of corrosion rate on the reliability index at reinforcement ratio equal to 1%.

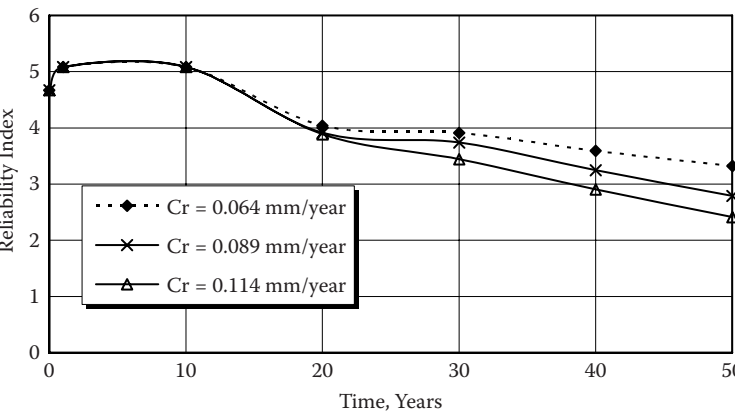


FIGURE 9.4 Effects of corrosion rate on the reliability index at reinforcement ratio equal to 4%.

study revealed that concentrically loaded reinforced concrete columns will be due for maintenance about 4 or 5 years from the initial time of corrosion with lowest steel ratio and higher corrosion rate. In cases of higher steel ratios and lower corrosion rates, this period may increase to about 15–20 years. Moreover, this study expects columns with eccentricity to increase the moment on the column, so the design should increase the percentage of steel. In such a case, it is expected that the deterioration of a structure and movement toward criticality will be after a couple of years in the case of lowest reinforcing steel ratio and very high corrosion rate. On the other hand, in the case of higher reinforcing steel ratio and lower corrosion rate, the maintenance will be due after about 6 years.

As we expected, the time required for repair is linked closely to the nature of a structure and the method of design, as well as to the importance of the building. For example, important vital structures, such as those in which nuclear activities take place, need protective measures with a time much different from that of other structures such as residential buildings.

We will apply the method of calculating costs through an example of protecting the reinforced concrete foundation in a petrochemical processing plant near the Red Sea. We will consider the same protection methods as those in Bentur, Diamond, and Berke (1997), who explained an economic study of a bridge surface for corrosion; because it always used salt to melt ice, the probability of chloride attack was very high. It is worth mentioning that the cost of repair and protection methods in the example is roughly based on the cost in Egypt. When applied in different countries, it will vary, but the price is stated just to perform cost comparison between the different ways of protection.

9.3 EXAMPLE

A petrochemical processing plant near the Red Sea has a 50-mm concrete cover for all foundations; the plant’s life span is 75 years. A ready-mix concrete with water-to-cement (w/c) ratio equal to 0.4 was used. The average daily temperature is about 26°C. The various types of protection include painting steel with epoxies, using silica fume, or using corrosion inhibitors. The initial cost estimate for protection by these methods is shown in table 9.1; the value of cost is calculated at the rate of the American dollar value per cubic meter. The cost of repair is assumed to be about \$200/m³ and is often the same procedure as those discussed in previous chapters: removing the cover and applying the normal repair procedure.

From table 9.1, corrosion inhibitor (1) is using an anodic inhibitor such as calcium nitrate with concentration of 10 kg/m³ and corrosion inhibitor (2) is using calcium nitrates with concentration of 15 kg/m³. The values of chloride ion content on the steel level are shown in this table. A discount rate of about 4% is imposed, as we mentioned

TABLE 9.1
Comparison of Different Ways of Protecting Steel

Cost (\$/m ³)	Method of protection
	No protection
40	Corrosion inhibitor (1)
50	Corrosion inhibitor (2)
80	Silica fume
55	Epoxy-coated steel bars
120	Silica fume + corrosion inhibitor (1)
130	Silica fume + corrosion inhibitor (2)
95	Epoxy-coated steel bars + corrosion inhibitor (1)
105	Epoxy-coated steel bars + corrosion inhibitor (2)
250	Cathodic protection system

earlier, with the assumption that repair will be sufficient for a period of 20 years only. All of these assumptions were taken into account when the comparison was made.

9.3.1 REQUIRED TIME TO START OF CORROSION

In the case of structures exposed to chloride, as in the previous example, the time required for the spread of chloride in the concrete until it reaches the steel is the time required for corrosion. Upon the chloride’s arrival, at a certain value limit, the corrosion will start. The assumption prevalence rate of diffusion is about $1.63 \times 10^{-12} \text{ m}^2/\text{sec}$ for concrete cast using a w/c ratio of about 0.4. It is assumed that the propagation rate is about $1.03 \times 10^{-12} \text{ m}^2/\text{sec}$ when silica fume is used. The propagation rate and its impact are as shown in figure 9.1, where the time needed to increase the content of the chloride on a steel bar is a result of the previous propagation of chloride inside a 50-mm-thick concrete cover.

Berke et al. (1996) offer computational methods for time of the beginning of corrosion. The results of the calculations are shown in table 9.2, which shows that the time needed to reach the amount of chloride to begin corrosion differs from the way for the protection of others. Note that there is no increase in the limits of chlorides that have occurred in the use of steel corrosion coated by epoxies or by using silica fume.

In the case of structures exposed to carbonation spread inside the concrete cover, several equations can calculate the time required for this propagation (see table 3.1 in chapter 3). The following equation can deduce the depth of the carbon transformation as follows:

$$d = A(B)^{-0.5} \tag{9.3}$$

TABLE 9.2
Time Required for Start of Corrosion

Method of protection	Chloride limit (kg/m ³)	Time to reach the limit	First repair time	Second repair time	Third repair time
No protection	0.9	17	20	40	60
Corrosion inhibitor (1)	3.6	37	40	60	
Corrosion inhibitor (2)	5.9	+75			
Silica fume	0.9	22	25	45	65
Epoxy-coated steel bars	0.9	17	32	52	
Silica fume + corrosion inhibitor (1)	3.6	51	54		
Silica fume + corrosion inhibitor (2)	5.9	+75			
Epoxy-coated steel bars + corrosion inhibitor (1)	3.6	37	52		
Epoxy-coated steel bars + corrosion inhibitor (2)	5.9	+75			

where A is a fixed amount depending on the permeability of the concrete as well as the quantity of carbon dioxide in the atmosphere and several other factors described in chapter 3.

$$A = (17.04(w/c) - 6.52) \cdot S \cdot W \quad (9.4)$$

where

w/c = the ratio of water to cement (less than 0.6)

S = the effect of cement type

W = the weather effect

These equations give the average depth of the transformation of carbon. Therefore, when calculating the maximum depth of the transformation of carbon, one should increase from 5 to 10 mm. $S = 1.2$ in the case of using cement it has 60% slag. $W = 0.7$ in the case of concrete protected from the outside environment.

9.3.2 TIME REQUIRED TO START OF DETERIORATION

The time required after the beginning of corrosion has already been discussed. As stated before, this time is about 3 years in the absence of a corrosion inhibitor and can extend up to 4 years when a corrosion inhibitor is added. On the other hand, when epoxy-coated reinforcing steel is used, the period is extended to 15 years. Note that the use of reinforcing steel coated by epoxy helps reduce the rate of corrosion in a clear reversal of the contraceptive use corrosion inhibitor as it is not actually affected in the reducing rate of corrosion.

Generally, the time of collapse of concrete cover from the beginning of corrosion depends on the rate of corrosion occurring in the steel reinforcement. Several studies have calculated the rate of corrosion. It was found that corrosion is closely related to relative humidity. As stated by Tutti (1982), in the case of steel corrosion due to carbonation, the corrosion rate is decreasing gradually at a relative humidity of 75% or less. The corrosion rate increases quickly when the relative humidity reaches 95%. It is noted that when the temperature degree decreases, the corrosion rate will decrease in temperature about 10 degrees, and the corrosion rate will decrease about 5–10%.

Broomfield (1997) stated that the rate of corrosion is affected by the relative moisture and a dry situation in which concrete is poured and the proportion of chlorides. Morinaga (1988) stated that the rate of corrosion was totally prevented when relative humidity was less than 45%, regardless of the chloride content and the temperature or oxygen concentration. Broomfield also stated that cracks in concrete occur when a lack of steel is about 0.1 mm and at less than 0.1 mm as well. That depends on the oxygen concentration and distribution, as well as the ability of concrete to withstand the excessive stresses. Sometimes, the placement of the steel bars causes cracks quickly when bars are adjacent to each other or in the corner. In practical measurement it is found that reduction in a section of about 10–30 μm results in a fragile layer of corrosion and is enough to cause concrete cracks.

The following equation of Kamal, Salama, and El-Abiary (1992) is used to calculate the time necessary from the beginning of corrosion for the emergence of the effects of corrosion on the concrete:

$$t_s = 0.08(C - 5) / (D.C_r) \tag{9.5}$$

where

- t_s = the time from beginning of corrosion until the concrete cover falls
- C = concrete cover thickness
- D = steel bar diameter
- C_r = corrosion rate units (mm/year)

The total time expected to perform the first repair depends on the time of the beginning of corrosion in addition to the time needed to increase the rate of corrosion to cause concrete deterioration and to perform the repair. This was clarified in table 8.2 in chapter 8.

9.3.3 COST ANALYSIS FOR DIFFERENT PROTECTION METHODS

An equation was applied for calculation of the present value of all methods of protection for steel reinforcement and is outlined in table 9.2. The different values of the present value of each protection method have been clarified in table 9.3, as have initial costs and the cost of repair. The total cost calculations of the current value are found in figure 9.5; three columns representing the cost of each method of protection are drawn in the graph.

TABLE 9.3
Cost Analysis for Different Protection Methods (\$/m³)

Protection method	Initial cost	First repair (NPV)	Second repair (NPV)	Third repair (NPV)	Total repair cost	Total cost
No protection	0	91.3	41.65	19.03	151.98	151.98
Corrosion inhibitor (1)	40	41.66	19.03		60.69	64.69
Corrosion inhibitor (2)	50	0				50
Silica fume	80	75	42.22		117.22	197.22
Epoxy-coated steel bars	55	57.03	26		83.03	138.03
Silica fume + corrosion inhibitor (1)	120	24.05			24.05	144.05
Silica fume + corrosion inhibitor (2)	133	0			0	133
Epoxy-coated steel bars + corrosion inhibitor (1)	95	26			26	121
Epoxy-coated steel bars + corrosion inhibitor (2)	105	0				105

TABLE 9.4
Comparison of Initial Cost, Total Repair Cost, and Total Cost for Different Methods

No protection	Initial cost	Total repair cost	Total cost
No protection	0	151.98	151.98
Corrosion inhibitor (1)	40	60.69	100.69
Corrosion inhibitor (2)	50		50
Silica fume	80	117.22	197.22
Epoxy-coated steel bars	55	83.03	138.03
Silica fume + corrosion inhibitor (1)	120	24.05	144.05
Silica fume + corrosion inhibitor (2)	133	0	133
Epoxy-coated steel bars + corrosion inhibitor (1)	95	26	121
Epoxy-coated steel bars + corrosion inhibitor (2)	105		105

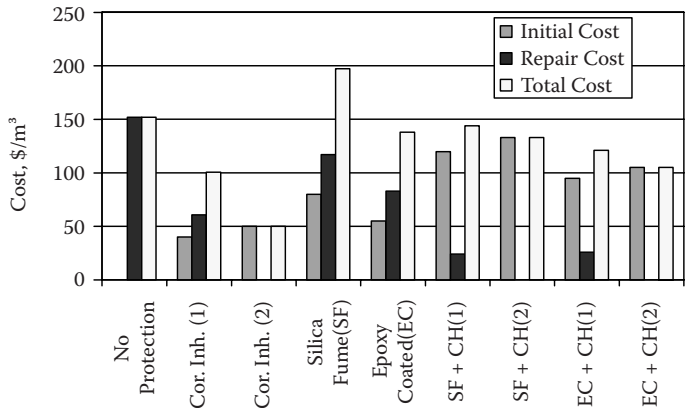


FIGURE 9.5 Economic comparison of different alternatives.

Adding the cathode protection method costs about \$500 to \$1500/m³; even taking into account that the lower cost of the cathode protection is \$500/m³, it will be much higher than the higher costs of other alternatives, as shown in the previous table. If the cathodic protection system is applied after 20 years, it would have a net present value (NPV) equal to around \$450 based on \$1000/m³ cost and no operating, maintenance, or repair costs for the next 55 years.

In the comparison between table 9.3 and table 9.4, one can choose the least costly method of protection and preferences. The previous example was a comparison among different methods of protection (see figure 9.5). However, the comparison must take into account the assumptions on which the calculations are based, including inflation and interest rates as well as labor and raw materials prices, which are recognized as initial costs.

9.4 REPAIR AND INSPECTION STRATEGY AND OPTIMIZATION

The decision-making methods of engineering projects have become the focus of numerous studies—as has their economic importance to the cost of the project as a whole—because the wrong decision could result in spending huge amounts of money to reverse it. Therefore, some studies have used decision trees to determine the appropriate time for repair, particularly in some bridges that need periodic inspection. When performing the maintenance plan strategy based on cost, we must consider all the factors that affect the corrosion (see chapter 5). Any method upon which a decision is based should define the time for regular maintenance (e.g., every 10, 15, or 20 years). This decision is reached first and foremost through the identification of the least expensive option.

Figure 9.6 summarizes the reliability of any structure during its lifetime. As one can see, the structure in the beginning will have higher capacity; the probability of failure value depends on the code or standard on which the structure design is based. After a period of time, the structure will deteriorate, thus increasing its probability of failure. After time Δt , when the inspection and repair have been performed, the structure will recover its original strength, as shown in the figure. After another period of time, the probability of failure will increase to a certain limit and then more maintenance will be performed, and so on.

Inspection alone does not improve reliability unless it is accompanied by a corrective action when a defect is discovered. Some policies and strategies used in a wide range of concrete structures, which have been programmed, include:

- monitoring until the crack depth reaches a certain proportion of the material thickness and then repairing
- immediately repairing on detection of indications of damage
- repairing at a fixed time (e.g., 1 year) after detection of indications of damage
- repairing as new (i.e., repair welding)

Generally, it is assumed that inspections are performed at constant time intervals, as shown in figure 9.7, since the inspection authorities often prefer a constant inspection interval to facilitate planning. From this point, the inspection intervals are chosen so that the expected cost of inspection, repair, and failure is minimized. While most

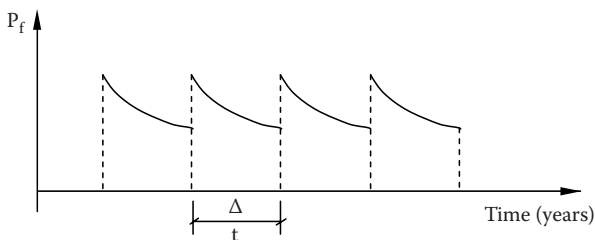


FIGURE 9.6 Concrete structure performance.

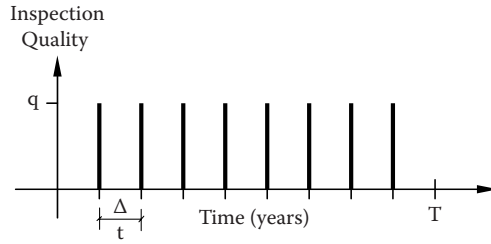


FIGURE 9.7 Inspection strategy.

concrete structure inspection techniques are visual or involve various NDT (non-destructive testing) inspection methods, the ability to detect damage depends on the quality of the inspection performed. A higher quality inspection method will provide a more dependable assessment of damage. No repair will be made unless the damage is detected.

9.4.1 REPAIR

Inspection may not affect the probability of failure of the structure. Following an inspection in which damage is found, a decision must be made regarding repair. This decision will depend on the quality of the inspection. With advanced inspection methods, even a small defect can be detected and repaired. Higher quality inspection may lead to higher quality repair, which brings the reliability of the structure closer to its original condition. Aging has an effect on the structure, so its reliability is decreased. In this chapter, it is proposed that, after inspection and repair, the structure capacity will be the same as in the design conditions, which are clearly shown in figure 9.4.

9.4.2 EXPECTED TOTAL COST

As mentioned in El-Reedy and Ahmed (1998), the first step is to determine the service life of the structure. Assume that it is 75 years and routine maintenance is scheduled once every 2 years (starting at $t = 2$ years and continuing until $t = 74$ years). Consequently, preventive maintenance work will be performed 37 times during the life of the structure. Therefore, the lifetime routine maintenance cost becomes:

$$C_{FM} = C_{m2} + C_{m4} + C_{m6} + \dots + C_{m74} \quad (9.6)$$

where C_{FM} indicates the total maintenance cost, and the total expected cost in its lifetime (T) is based on the present value worth. The expected lifetime preventive maintenance cost becomes:

$$C_{IR} = C_{IR2} \frac{1}{(1+r)^2} + C_{IR4} \frac{1}{(1+r)^4} + C_{IR6} \frac{1}{(1+r)^6} + \dots + C_{IR74} \frac{1}{(1+r)^{74}} \quad (9.7)$$

where C_{IR} is the periodic inspection and minor repair and r is the net discount rate of money.

In general, for a strategy involving m lifetime inspections, the total expected inspection cost is:

$$C_{ins} = \sum_{i=1}^m C_{ins} + C_R \frac{1}{(1+r)^{T1}} \quad (9.8)$$

where

C_{ins} = inspection cost based on inspection method

C_R = repair cost

R = net discount rate

Finally, the expected total cost, C_{ET} , is the sum of its components, including the initial cost of the structure and expected costs of routine maintenance, preventive maintenance (including inspection and repair maintenance costs), and failure. Accordingly, C_{ET} can be expressed as:

$$C_{ET} = C_T + (C_{ins} + C_R)(1 - P_f) + C_f \cdot P_f \quad (9.9)$$

The objective remains to develop a strategy that minimizes C_{ET} while keeping the lifetime reliability of the structure above a minimum allowable value.

9.4.3 OPTIMIZATION STRATEGY

To implement an optimum lifetime strategy, the following problem must be solved:

$$\text{Minimize } C_{ET} \text{ subjected to } P_{f\text{life}} \leq P_{\max}$$

where P_{\max} = maximum acceptable lifetime failure probability. Alternatively, consider the reliability index:

$$\beta = \phi^{-1}(1 - P) \quad (9.10)$$

where ϕ is the standard normal distribution function. The optimum lifetime strategy is defined as the solution of the following mathematical problem:

$$\text{Minimize } C_{ET} \text{ subjected to } \beta_{life} \geq \beta_{\min}$$

The optimal inspection strategy with regard to costs is determined by formulating an optimization problem. The objective function (C_{ET}) in this formulation is defined as including the periodic inspection and minor joint repair cost, as well as the failure cost, which includes the cost of major joint repair. The inspection periodic time (Δt) is the optimization variable constrained by the minimum index, β , specified by the code and the maximum periodic time.

The optimization problem may be mathematically written as:

Find Δt , which minimizes the objective function:

$$C_{ET}(\Delta t) = (C_{IR})(1 - P_f(\Delta t)) \left(\frac{(1+r)^T - 1}{\left((1+r)^{\Delta T} - 1 \right) (1+r)^T} \right) + \quad (9.11)$$

$$C_f P_f(\Delta t) \left(\frac{(1+r)^T - 1}{\left((1+r)^{\Delta T} - 1 \right)^T (1+r)^T} \right)$$

subject to $\beta(t) \geq \beta^{\min}$, $\Delta t \leq T$,

where

C_{IR} = the periodic inspection and minor repair cost per inspection

C_f = the major repair cost

i = the real interest rate

β^{\min} = the minimum acceptable reliability index

C_{IR} and C_f are assumed constant with time.

Even though the failure cost is minimized (as part of the total cost), it is often necessary to constrain the reliability index to fulfill code requirements. Since the time period for the proposed repair is T , n is the total lifetime of the building, and C_{IR} is the cost of inspection and repairs, they are costs resulting from the collapse. The value of the expected cost at each period of time is a curve that follows that in figure 9.8, which defines the period of time required for the process of periodic maintenance, which is achieved less expensively, as in the curve that follows.

The period of time required for the process of the periodic maintenance, which is achieved at the highest cost, is Δt . The previous equation generally can be used for comparison between different types of repair with different time periods and different costs. It is calculated by adding the initial cost of the repair and the design curve.

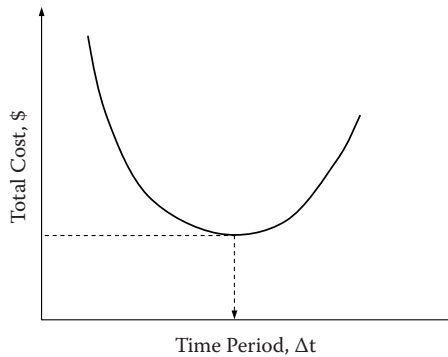


FIGURE 9.8 Optimization curve to obtain optimum maintenance time period.

TABLE 9.5
Relation between Structure Lifetime and Probability of Failure

Life (years)	0	10	20	30	40	50	60
Probability of failure	0	9%	25%	50%	75%	91%	100%

The decision can be determined in terms of the expected cost and risk of collapse, and is calculated by using different methods to calculate the possibility of the collapse of the concrete member, or the use of the approximate method as shown in table 9.5. For the possibilities which are imposed at the beginning of the age of structure, the probability of collapse at the age of zero and the structure lifetime is assumed to be 60 years old. The possibility of the collapse is 100% at this age, and then the probability of collapse at every period is calculated as shown in table 9.5.

Some special software programs for the management of bridge systems (BMS), as identified by Abd El-Kader and Al-Kulaib (1998), define the time required to conduct the inspection and maintenance of each bridge. They consider the rate of deterioration in bridges with the lifetime of the structure. One such program includes a previous section that has been studied, a limitation on the time required for periodic inspection and maintenance work, and how to determine the cost, which depends on the budget planning for the repair of bridges. The program was developed by the Federal Highways Association (FHWA).

The maintenance plan and its implementation depend on the criticality and the importunity of the building. In order to proceed with maintenance, it is necessary to consider the sequences of failure. For example, concrete columns and concrete slab differ. Cost of repairing the two members may be the same or, in some situations, the repair of the concrete slab on the grade will cost more than repairing the columns. Which should be repaired first? Obviously, repair will start on the concrete columns because failure will consequently cause bad consequences from an economic point of view. Therefore, as we mentioned earlier, the previous equation is a general equation, so all the factors that affect decision making must be taken into consideration.

Generally, to proceed with maintenance on a regular basis in a certain defined time period is called preventive maintenance. It is important because it extends the structure's lifetime and maintains real estate that represents economic wealth for a country—especially in developing countries, in which all the materials used in the construction industry are imported. Also, some countries have a huge investment in their coastal cities, which are highly vulnerable to the process of corrosion due to chloride effects.

On the other hand, the amount of carbon dioxide in the atmosphere in cities is increasing due to huge numbers of vehicles moving daily; this increases the corrosion of reinforcing steel bars. In addition, there are problems such as sanitation and drainage systems in houses, considered one of the key factors in corrosion of steel reinforcement of the bathroom's slab. This exposes the weakness of the structure and is a serious issue for the whole building. Therefore, regular maintenance and development of an integrated plan for the maintenance of each structure are very important to preserving a structure's lifetime and therefore to the national wealth of a country.

9.5 MAINTENANCE PLAN

In any organization, the maintenance team is responsible for maintaining the structural reliability of the buildings. The responsibility of this team is clearly in the ministry of transportation that is responsible for bridge maintenance. For example, in the United States, the famous Golden Gate Bridge is maintained by a team that regularly starts repainting the first section of the bridge as soon as it has finished painting the last section.

When we discussed the time spent performing maintenance earlier, we considered the probability of structural failure. From a practical point of view, it is complicated to calculate this accurately as it is needed for research rather than for practical issues. The calculation of the probability of failure and its consequences to obtain the structural risk is called the qualitative risk assessment. The other popular method is the quantitative risk assessment; it is easy and can be handled through the maintenance team without outside resources.

9.5.1 ASSESSMENT PROCESS

The assessment of a concrete structure depends on its structural type, location, and existing load and operation requirements. For studying the risk assessment, different critical items that have an economic effect must be considered. The general definition of risk is summarized in the following equation:

$$\text{Risk} = \text{probability of failure} \times \text{consequences}$$

To define the risk assessment of any concrete structure, the factors that may affect the business's economics are worth considering. The structural risk assessment is represented by the probability of failure and its consequences; the whole building, part of the building, or only a concrete member may fail.

The first step to define and calculate the quantitative risk assessment is to hold a meeting with the maintenance team, and maybe with a consultant engineering company when huge structures are involved. Within this meeting, which will take several hours, different discussions define the factors that affect the probability of structural failure, such as corrosion on the steel reinforcement bars, overload compared to the design, new data revealing lower concrete strength than the design and project specifications called for, and the redundancy of the structure itself. The required data about the corrosion of steel bars, concrete compressive strength, and other factors have been discussed before and can be obtained by collecting the data and performing the visual inspection, as well as performing detailed inspection using measurement equipment such as ultrasonic pulse velocity, rebound hammer, or other available techniques. The factor that has the most influence on estimating the probability of structural failure is the structure's redundancy, which can be explained by figures 9.9 and 9.10.

A beam needs to be designed and the following two solutions are offered: (1) a structural system to be fixed at the two ends or (2) a structural system hinged at the two ends. Which system should be chosen? Take 5 minutes to think about this.



FIGURE 9.9 Comparison of structural redundancy.

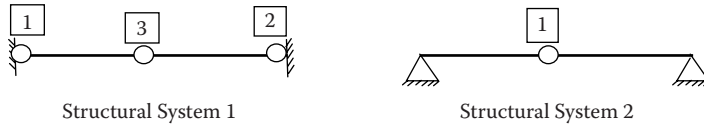


FIGURE 9.10 Comparison of structural redundancy.

Many factors control a decision in selecting a suitable system. The following give the advantages and disadvantages for these two systems:

- Structural system 1:
 - The beam cross-section will be smaller.
 - The connections will be big and complicated as they have shearing force and moment.
 - It is reasonable from an architectural point of view.
 - The construction is complicated in connection.
- Structural system 2:
 - The beam cross-section will be big.
 - The connection will be small as it is designed to shear force only.
 - It is easy to construct because the connection is simple.

After discussing these two systems, maybe the decision will be for in-house engineers and workers to do the construction. To avoid any problems, the simple beam option will be chosen. This type of thinking is always happening as the maintenance point of view is forgotten. Figure 9.10 shows the steps of collapse failure.

Structural system 2, which is the simple beam, assumes that the load is increased gradually and the beam can accommodate the load until a plastic hinge is formed in the middle of the beam at the point of maximum bending moment; then, collapse will occur. On the other hand, for structural system 1 that is fixed at the two ends, when the load is increased gradually, the weaker of the left or right connection will fail first. As shown in the figure, the plastic hinge will form on the left connection and increase the load; the other connection will be a plastic hinge, so it is now working as a simple beam. By increasing the load, plastic hinge 3 will be formulated and then collapse will occur.

From the preceding discussion, one can find that structural system 1 will take more time to collapse because it fails after three stages. Structural system 2 fails in the first stage, so system 1 is more redundant than system 2. Moreover, when a comparison is made among different reinforced concrete members, such as slabs, beams, columns, and cantilevers, one can learn that some members are more critical than others. A cantilever is most critical because any defect in it will have a high deflection and then failure. In addition, when a column fails, the load is distributed to

other columns until the whole building fails. Thus, the column has low redundancy and is very critical because any failure on it will result in the whole building failing. However, the cantilever failure will be a member failure only, so we can go through the consequences from this approach.

In the case of beams and slabs, when a slab is designed, a simple beam is assumed and the maximum moment is calculated in the middle. Then, the concrete slab is designed by choosing the slab thickness and the steel reinforcement. The selected steel reinforcement will be distributed along the whole span. Theoretically, by increasing the load, the failure will be at a point but actually the surrounding area will carry part of this load, so the redundancy of the slab is very high. Some research has mentioned that the reinforced concrete slab can accommodate loads that are twice the design load.

For the structure as a whole, we can use pushover analysis to obtain the redundancy of the structure. This analysis is nonlinear and now is available for any structure analysis software in the market. From this analysis, one can obtain how much more load than the design load a structure can carry until failure. In addition, one can know the location of the first plastic hinge to formulate and, from this, one can know the critical member in a structure.

Therefore, it is necessary to put a maintenance plan in place for a concrete structure. The first step is to discuss all the structural parts that need to be inspected and repaired and capture this information in a table. The team will put all the factors that affect the probability of structural failure into this table. Different factors will be assigned different values. For example, in the case of a redundancy factor, values will run from 1 to 10, and a column will have a value of 10 and a slab will have a value of 2. All the members of the team must agree on these numbers, based on their experience. The same values or higher values will be repeated for other factors, such as age, the engineering office that produced the design, the contractor who handled the construction, and the code used in design. As we discussed before, in some countries those responsible for construction have used seawater in mixing water, and some codes in the past agreed to use 6-mm-diameter steel in stirrups. Therefore, if corrosion occurred, these stirrups could not be seen because complete corrosion would have taken place.

Table 9.6 contains the probability of structural failure factors, but, when this table is used, it must match with structural requirements. The table is not limited to these factors only; it is a simple example of calculating the quantity of the probability of failure. However, when a meeting is held, it is necessary to develop criteria about the number. For example, consider an engineering office that produces the design. If it is professional and well known in the market and one has had no problems with the firm, then lower values should be assigned. For an incompetent firm with which one has had problems in the same type of structure, a higher value is assigned. This procedure will apply also to the contractor.

The code value comes from the experience of the team considering problems not only for its own buildings, but also for buildings throughout the country. Therefore, in some complicated structures or huge facilities it is worth contacting a competent engineering office for a maintenance plan.

If one conducts no inspections, the building is a “black box” and should be given the highest value. The things that one knows are bad are less critical than those

TABLE 9.6
Factors’ Effects on Probability of Structure Failure

Structure	Redundancy (1–10)	Age (1–10)	Designer (1–10)	Contractor (1–10)	Code (1–10)	Last inspection (1–10)	Total score
S1	8	10	6	6	9	10	49
S2	5	2	7	3	3	10	30
S3	3	5	8	10	5	2	33
S4	7	4	2	5	5	1	24
S5	1	3	4	2	5	10	25
S6	3	1	4	5	6	7	26

TABLE 9.7
Consequence Weight

Structure	Impact on person (1–10)	Impact on cost (1–10)	Impact on environment (1–10)	Impact on repetition (1–10)	Total score
S1	8	10	6	6	30
S2	5	2	7	3	17
S3	3	5	8	10	26
S4	7	4	2	5	18
S5	1	3	4	2	10
S6	4	5	3	5	17

things about which one knows nothing. Then, it is necessary to go through the consequences with the same approach, considering the economic impact. Economic impact is the answer to the question of what the effect will be from an economic point of view if the whole structure or part of it fails (see table 9.7).

There are many hazard factors that will guide structure risk assessment. The first factor is the location itself, as well as its expenses and if it carries a processing facility for industry that must be taken into consideration because the failure may cause hazards or stop production. For example, consider the foundation under machines; any failure in it will influence the performance of the machine and will be the main cause for the processing plant’s shutting down and stopping production. We calculate the weight of the consequences by the same procedure as that for probability of failure. Now that we have the weight of the probability of failure and the consequences, we calculate the risk assessment from table 9.8.

9.5.2 RISK-BASED INSPECTION MAINTENANCE PLAN

After we calculate the risk assessment as shown in the previous table, we classify the maintenance plan. The top third of the top risk structures will be red, the second third will be yellow, and the remaining third will be green; the priority list will be

TABLE 9.8
Risk Weights for All Structures

Structure	Probability of failure	Consequences	Risk
S1	49	30	1470
S2	30	17	510
S3	33	26	858
S4	24	18	432
S5	25	10	250
S6	26	17	442

TABLE 9.9
Structure Priority List

Structure priority	Color code	Risk value
S1	Red	1470
S3	Red	858
S2	Yellow	510
S6	Yellow	442
S4	Green	432
S5	Green	250

as shown in table 9.9. From this table it can be seen that the structures S1 and S3 are considered the critical structures upon which inspection would start. Structures S2 and S6, as the second priority, would be inspected after the first two structures. The last structures, S4 and S5, will be at less risk, so inspection would not be needed at this time.

As shown in the table, this simplified method can be used to plan maintenance for inspection and repair, taking into account that the budget is a very important factor in the maintenance plan. From the previous example, the budget may be enough this year to inspect structure S1 only, so it will be necessary to plan for the following years—maybe for the next 5 years. After the inspection is performed, the data will be analyzed through the system as in the flow chart in figure 9.11.

Nowadays, for international organizations with a number of different buildings and structural elements, the integrity management system is very important to maintaining structures along their lifetimes and accumulating historical data. The closed loop of a structure risk assessment, an inspection program, a repair program, and reassessment is crucial to maintaining the structure in a good condition and as a reliable match with the operations requirements. The total quality control management system is very important for reviewing design, construction, and maintenance to be matched with operational requirements in cases of proposed development of or extension to a building or increasing its load. Therefore, it must be considered by a system of change management procedures to upgrade the previous risk-ranking table.

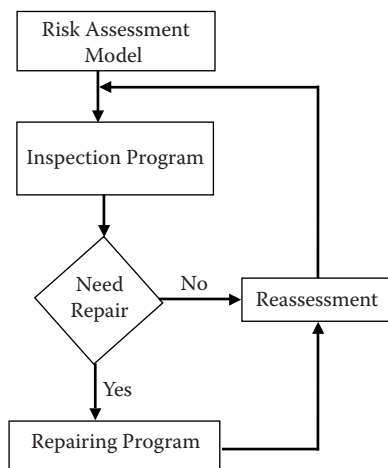


FIGURE 9.11 Risk assessment system.

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STEEL-REINFORCED CONCRETE STRUCTURES

Carrying a billion-dollar price tag, corrosion of reinforced concrete is the enemy of every country's investment in real estate. The widespread and long-term use of reinforced concrete makes its correct and proper examination, maintenance, and repair paramount. ***Steel-Reinforced Concrete Structures: Assessment and Repair of Corrosion*** explains the corrosion of reinforced concrete from a practical point of view, highlighting protective design and repair procedures.

The book begins with a discussion of the corrosion phenomena, the effect of concrete properties on corrosion, and the precautions available in the construction stage to mitigate corrosion. It covers the theoretical and practical methods in evaluating the concrete structures and new, practical methods to protect steel reinforcement. The book also includes methods established in the last decade that provide new ways of protecting steel-reinforced bars and the traditional and advanced repairing methods. The author explains the importance of implementing an integrity management system to provide a comprehensive maintenance strategy and concludes with coverage of the traditional, time-tested, and advanced repair techniques. A special feature is a chapter focusing on the advance maintenance plan philosophy and risk-based maintenance for reinforced concrete structures.

The author examines economic analysis procedures and the probability of structural failures to define structure risk assessment. He covers precautions and recommendations for protecting the reinforced concrete structures from corrosion based on codes and specifications. He uses case histories from all over the world to demonstrate the widespread application and range of advanced repair techniques and presents a practical guide to the maintenance of concrete structures. The book provides procedures for corrosion diagnosis and determining the appropriate methods for repair, as well as economic models for on-site decision making.



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